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1923

REPORT

OF

HYDRAULIC POWER COMMITTEE

TECHNICAL NATIONAL SECTION

Including

REPORT ON PENSTOCK DESIGN

Prepared Jointly With

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CONTENTS

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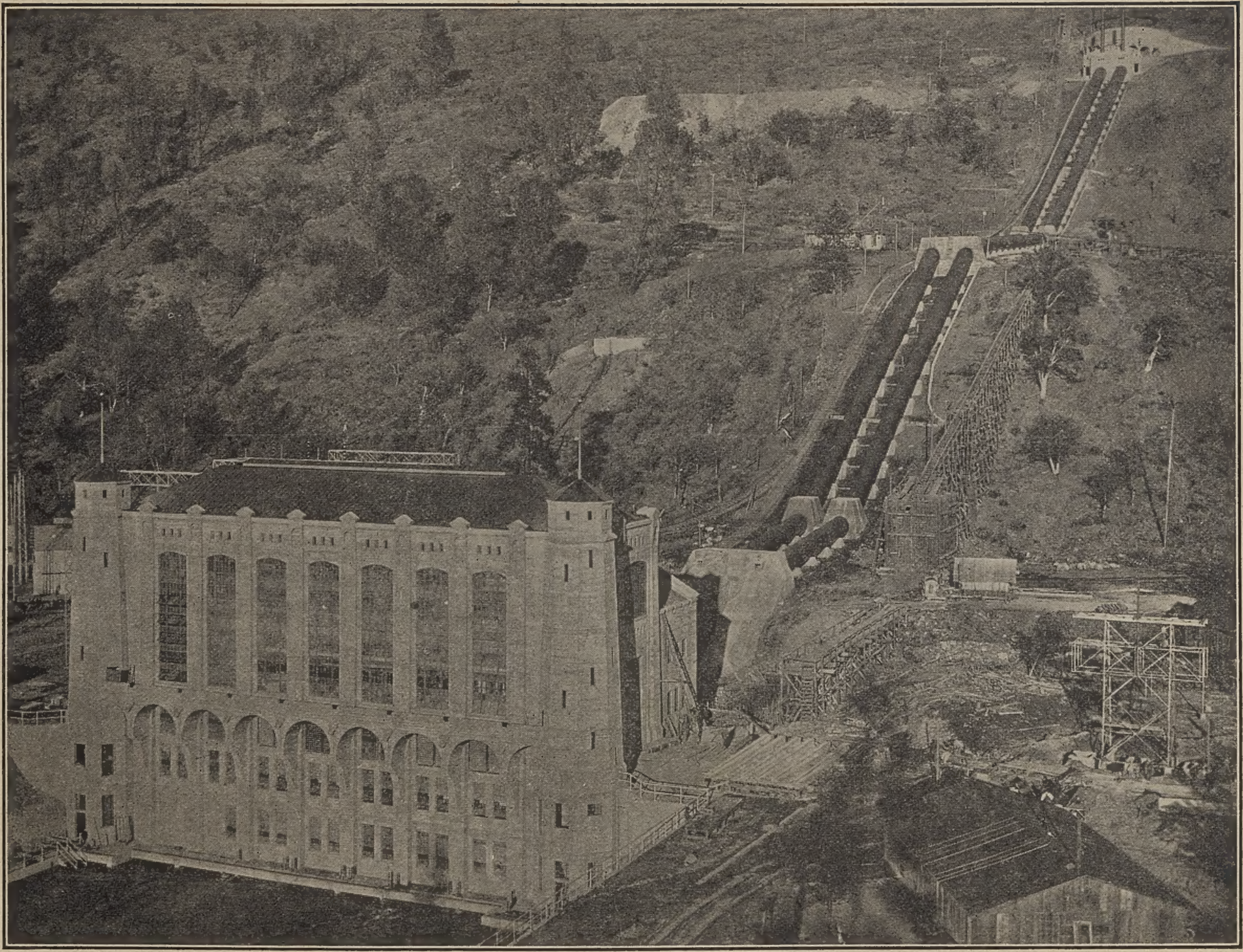

Pit No. 1—Power House and Penstocks, Pacific Gas & Electric Company.

23-32

REPORT OF THE HYDRAULIC POWER COMMITTEE

The third annual report of the Hydraulic Power Committee is submitted herewith. At the beginning of the year's work it was decided by the members of the Committee to restrict its activities to a very few subjects in order that those selected might be covered completely. On account of the diversity of interest of the various members of the Committee, it has been difficult to follow this program, but we believe the results of the year's work will show that the policy is a wise one and, if followed for a few consecutive years, will result in the compilation of a great deal of technical information which will be of great value to the industry.

Respectfully submitted,

HYDRAULIC POWER COMMITTEE

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PENSTOCK DESIGN

JOINT REPORT OF

SUBCOMMITTEE OF HYDRAULIC POWER COMMITTEE AND PACIFIC COAST ELECTRICAL ASSOCIATION

JANUARY, 1923

In this report an effort has been made to collect as much data as possible on the practice of various companies and engineers in the matter of design of penstocks. It has been realized by those engaged in the work that there is a great diversity of opinion as to the proper method to pursue in the design of the many important elements entering into a complete penstock. It was therefore thought that a report of this sort might clear up some of these disputed points.

As was to be expected, the major portion of the report represents Western practice, as most Eastern plants are of low head where the penstock is of minor importance. When a high head is involved, the penstock must be designed with the utmost care to insure safety and economy.

It was hoped, at the beginning, that this investigation would result in a set of specifications and methods that could be adopted as standard practice. On account of the ground to be covered and the lack of time for consultation with the many members of the Committee, much yet remains to be done to achieve this result. It is believed, however, that a goodly amount of valuable data has been collected, and it is hoped that the report will be of some value to those engaged in hydro-electric design.

Economic Size

It is practically the universal practice to design a penstock so that its "annual cost" is a minimum. This annual cost is made up of the interest and depreciation on the value of the penstock installed plus the value of the power lost in friction. In some cases the cost of pier anchors, excavation and backfill is included with the cost of the pipe proper, but in general the cost is limited to the value of the pipe itself, in place; the other items should be included only if they are found to vary materially with the pipe diameter. The value of the power lost in friction is taken either as the cost of the power at the power house or, in some cases, as the selling price of the power.

Many methods have been devised for arriving at the pipe diameter that will give the minimum annual cost. Prof. W. F. Durand in his "Hydraulics of Pipe Lines" outlines a method that has been used by some companies. Another method is given by D. W. Mead in his "Water Power Engineering"—second edition.

One company uses the familiar rule that the economic size will be that which results in the value of the power lost in friction being four-tenths of the fixed charges on the pipe. In cases where the pipe is buried to half its diameter, this rule is modified so that the "yearly value of frictional energy loss shall equal 4/10 of the annual cost of the pipe plus 3/10 of the annual cost of the excavation and backfill."

Another interesting method involves the plotting of a curve, the abscissæ of which represent size of conduit and the ordinates represent cost in dollars. From this curve may be plotted another one which shows increment in cost against increment in size. As the size of the conduit increases there is a gain in the power developed, which may be calculated from the assumed coefficients of friction. The increment in cost of conduit may then be related to the increment in power to obtain the cost per horsepower expended as the size of the conduit is increased. It is then possible to plot on a curve sheet, with size of conduit as abscissæ and cost per horsepower as ordinates, a curve which shows the increment of cost divided by the increment in power. Such a curve enables one to tell at a glance the price in dollars per horsepower that would be paid for any increase in the size of conduit. This price, of course, may be adjusted to suit any particular proposition.

Another graphical method involves the plotting of the total annual cost per foot of pipe as ordinates and length of pipe in feet as abscissæ. These curves are plotted for several different diameters. It is seen that the area under one curve represents the total annual cost of a pipe of a given diameter. Since it is desired to have the annual cost of the whole line a minimum, the combination of diameters that will give the minimum area under the curves will be the most economical pipe to use.

Where the flow through the penstock is a variable amount during the year, all calculations for economic size can be greatly simplified by using a constant mean flow that will give the same total *power loss* due to friction. This equivalent mean flow is the cube root of the mean of the cubes of the ordinates of the curve of variable flow.

Maximum Velocity

The general tendency seems to be to limit maximum velocities to 10 or 12 feet per second. This is probably due to the fact that the majority of plants are of moderate heads. With high head plants the maximum velocity will be nearly double these values if the pipe is designed on an economic basis. Several high head western plants have velocities from 15 to 21 feet per second and pipe lines are now being designed for even higher velocities. This is more nearly in accord with the practice in Europe and Japan. Many pipe lines of high velocity have been in operation several years and no deterioration of the pipe, or other objectional result, has been noticed.

Head Used in Determining Thickness

General practice is to design the penstock for the static head, allowing the factor of safety to provide the margin of strength required to withstand excess pressures due to water hammer.

FIG. 1—TYPICAL PENSTOCK INSTALLATION, KERN RIVER NO. 3 PLANT, SOUTHERN CALIFORNIA EDISON COMPANY.

In some instances the head used in determining thickness of the pipe is the static head plus the computed water hammer due to normal governor stroke during a total shut down. Probably the best known method of determining water hammer is that developed by Norman R. Gibson and fully described by him in a paper No. 1439, published in Vol. LXXXIII of the Transactions of the American Society of Civil Engineers.

Another method has been devised by H. C. Vensano and is described by him in an article entitled "Pulsations in Pipe Lines," published in Vol. LXXXII of the American Society of Civil Engineers.

It is considered good practice to increase the thickness of the pipe by a constant quantity throughout its entire length to provide against decrease in thickness due to corrosion.

Factor of Safety

With very few exceptions it is believed advisable to have the factor of safety on the elastic limit of the steel instead of the ultimate strength. It is the general belief that a factor of safety of two based on the elastic limit is proper for all ordinary cases. If the pipe line is long and the velocity high it is recommended that special consideration be given to the factor of safety in order to provide for possible water hammer. It is the general opinion, however, that for the majority of lines, a factor of two will provide sufficient safety against failure due to water hammer. It is the consensus of opinion that the maximum stress that can occur in the pipe should not reach the elastic limit.

One advantage in using a factor of two on the elastic limit rather than the more common factor of four on the ultimate strength is that this results in a higher working stress and consequently a lighter pipe. The reason for this is that the steel generally used for welded pipe has an elastic limit considerably above half the ultimate strength.

Many actual tests of steel for welded pipe show the elastic limit about 66 per cent of the ultimate. It is therefore apparent that with this steel a factor of safety of two on the elastic limit would result in a saving of about 30 per cent in weight over a factor of four on the ultimate strength.

Minimum Thickness of Steel

There is no general agreement as to the minimum thickness of plate to use at the upper end of a line. The following table gives data on large pipes of thin plate that have been in operation many years and have given entirely satisfactory service. These pipes are much thinner than would be called for by the common shop rule that the thickness should not be less than 1/200 of the diameter.

Location	Diameter	Thickness
Jawbone Siphon	10 ft.-0 in.	1/4 in.
Nine Mill Siphon	9 ft.-6 in.	1/4 in.
Soledad Siphon	11 ft.-0 in.	1/4 in.
Kern River No. 3 Siphon	9 ft.-6 in.	3/8 in.
Hat Creek Penstock	10 ft.-0 in.	5/16 in.

Some companies have in their later work adopted minimum thicknesses considerably greater than those shown above. There is also a desire on the part of manufacturers of welded pipe to do likewise. It would appear to be good practice to use a minimum thickness of 3/8 in. to 1/2 in. for welded pipes over 60 in. in diameter. For riveted pipes minimum thicknesses are recommended as follows: 1/4 in. for 72 in. diameter; 5/16 in. for 84 in. diameter; 7/16 in. for 120 in. diameter. A verv valuable discussion of this point will be found in the manufacturers' statements at the end of this report.

Stiffening Rings

Stiffening rings have not been used extensively on penstock lines, but it is believed that they would be of value in riveted pipes exceeding ten feet in diameter, by making it possible to use thin plates without danger of collapsing the pipe. Experience on one large line indicates that the change in shape of the pipe during transportation was sufficient to open up the shop caulking enough for a slight leakage to develop when the pipe was filled. Where transportation conditions are not difficult and where the pipe is properly blocked so as to retain its circular shape, the above condition would not necessarily result. Had stiffening rings been used, however, they would have prevented the distortion and resulting opening of the seams.

Buried vs. Exposed Pipe

Recent practice is decidedly in favor of leaving the penstock exposed. In deciding whether a pipe should be buried or exposed the following factors should receive consideration:

1. *Danger From Slides*

   Where the topography is such that there is a possibility of slides (either snow, rock or earth) across the pipe line, it is advisable to bury the pipe.
2. *Profile of Pipe Line*

   Where the profile of the country is such that there are a great number of angles which cannot economically be limited, it might be advisable to bury the pipe and eliminate or reduce the number of expansion joints.
3. *Size of Pipe*

   For small pipe lines it will not usually be economical to construct the piers, though it will be economical for large pipe.
4. *Type of Ground*

   Where material is expensive to excavate even for small pipe, it will prove economical to support the pipe above ground.

Advantages and Disadvantages of Exposed Pipe Lines

The advantages of exposed pipe aside from possible economy of first cost, are

1. Greater accessibility for construction, inspection, caulking, repairs, painting and general maintenance.
2. Longer life of pipe line.

The disadvantages are

1. Probability of formation of ice in freezing weather.
2. Necessity for expansion joints to allow for temperature stresses.

For pipe lines of the size used for penstocks the advantages are therefore generally on the side of the exposed pipe line.

Experience with Buried Pipes

With some old pipe lines it was the practice to excavate first a trench, then rivet the joints together consecutively, supporting them temporarily on wooden blocking raising the pipe about six inches above the bottom of the trench. After testing, the trench was backfilled with earth, completely covering the

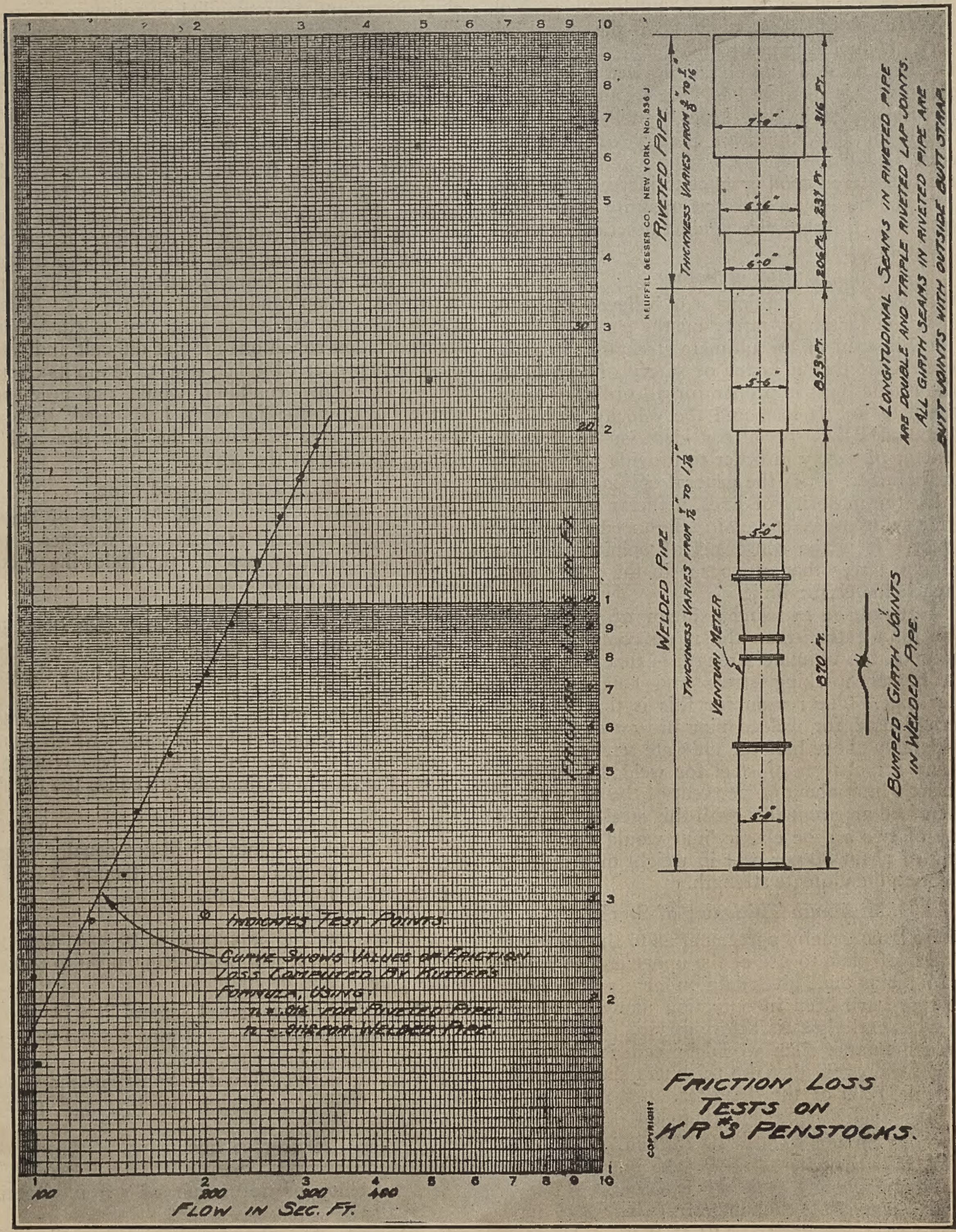


FIG. 2—FRICTION LOSS—TESTS ON KERN RIVER NO. 3 PENSTOCKS.

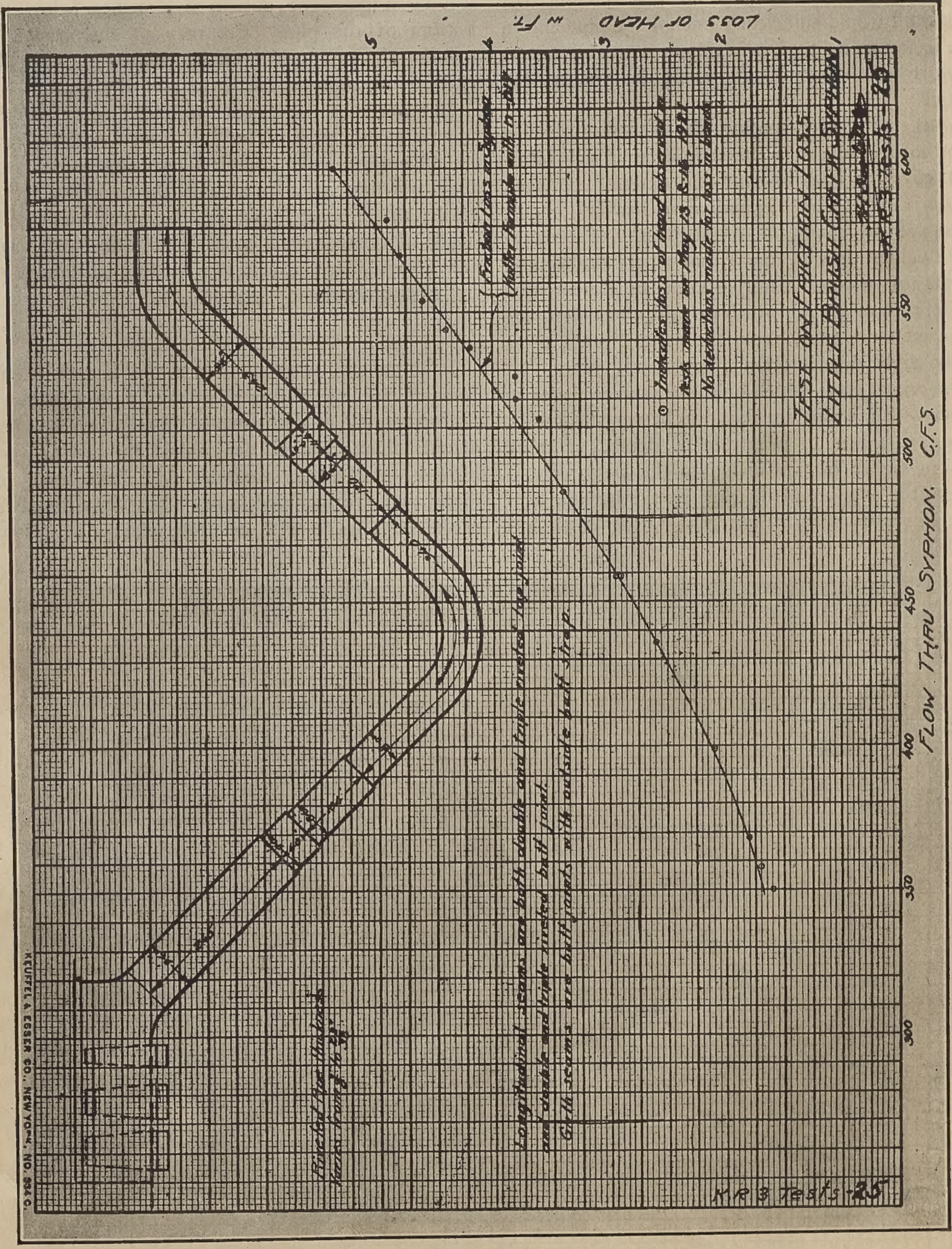


FIG. 3—FRICTION LOSS—TESTS ON LITTLE BRUSH CREEK SIPHON.

pipe. A certain amount of tamping was done underneath to supplement the blocking and take the weight after the wood has decayed. It was thought that the backfill would sufficiently anchor the pipe and in some instances it has, but in others it was necessary to resort to concrete blocks as in the present day practice.

For the protective coating, the pipe was given a dip in hot asphaltum in the shop after it was fabricated, covering both interior and exterior surfaces.

From time to time, examination has been made to observe the condition of the paint and its protective value. It was found that where the backfill consists of porous material such as decomposed granite, rock or gravel, which permits a free drainage of the moisture away from the painted steel, there is only slight deterioration after twenty years.

Impervious clays which hold the surface water in contact with the pipe are found to be very injurious, causing rust pits to penetrate to depths of more than one-eighth of one inch into the steel plate.

As to the support of the pipe by the backfill, evidence has shown that even though the earth has been tamped as previously explained, it has been found that settlement takes place to the extent of leaving a space of one to five inches below the bottom of the pipe. Contrary to the prevailing opinion, the surface run-off water does not pack the earth firmly around the bottom of the pipe.

In other lines, the trench was backfilled only to the middle of the pipe. This has been found to have no advantages over complete covering aside from the lower first cost. In all soils the corrosive action seems to be most rapid on the sides and for a depth of about twelve inches below the surface of the ground, due apparently to the conditions of extreme variations of heat and cold and moisture.

Coefficient of Friction

It is general practice to use the Kutter Chezy formula for calculating the friction in penstocks. Unfortunately, the information available regarding actual friction values determined by test is very meagre. All readily available data have been collected and are included here. Only data based on actual tests made with accurate measuring instru-

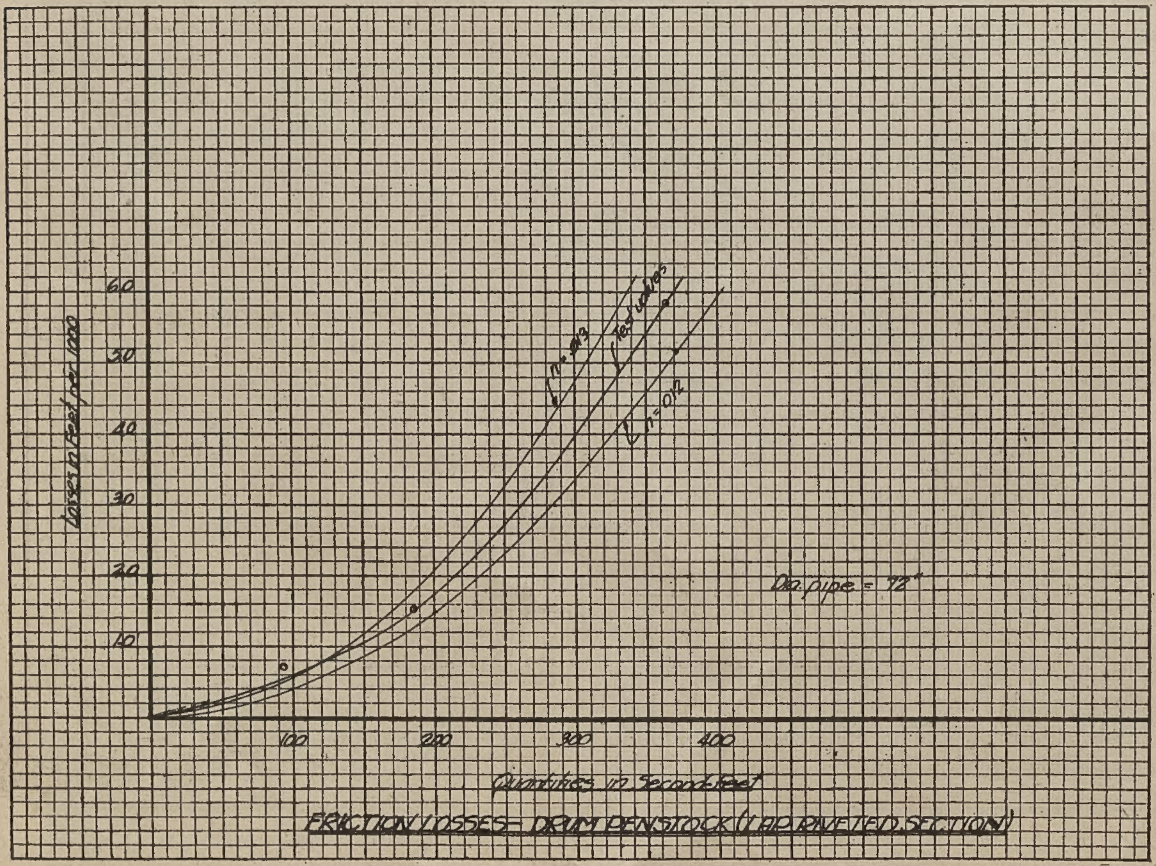


FIG. 4—FRICTION LOSS—DRUM PENSTOCK (RIVETED SECTION).

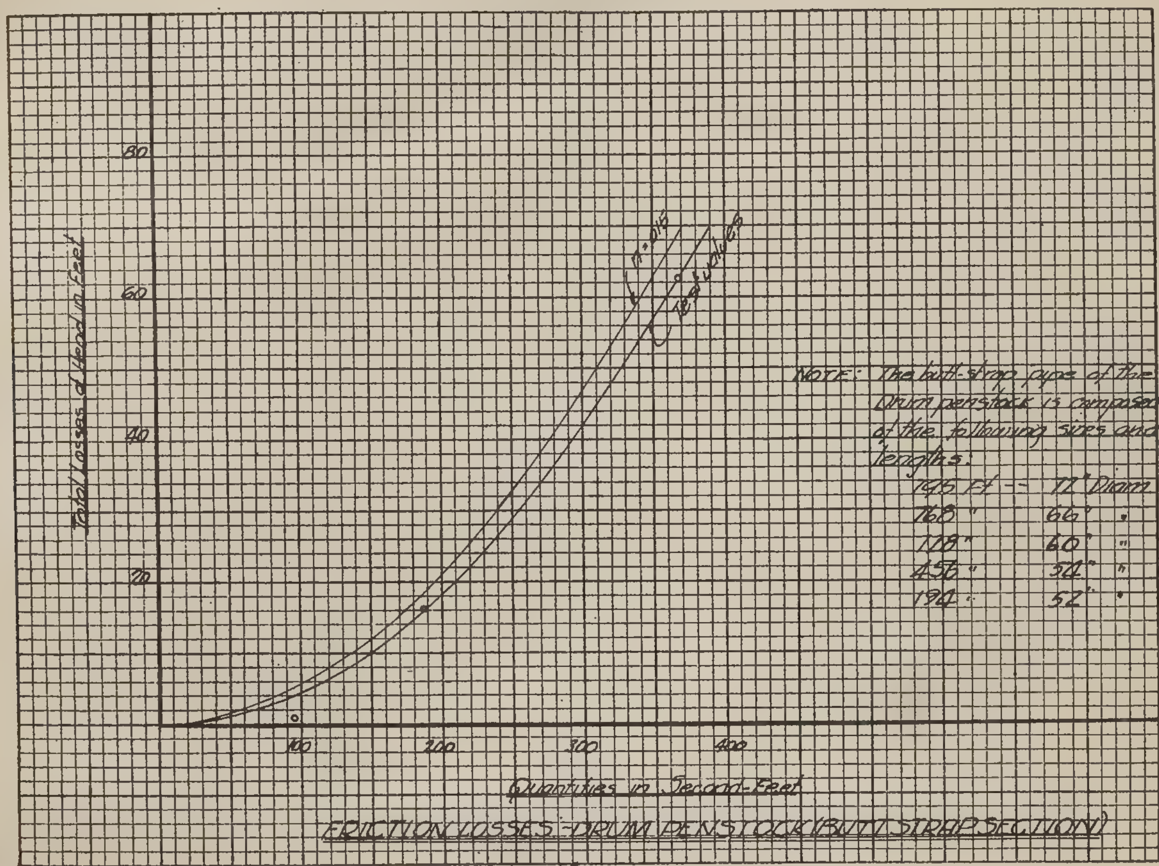


FIG. 5—FRICTION LOSS—DRUM PENSTOCK (BUTT STRAP SECTION).

ments have been collected, and it is felt that these results are reliable and representative.

It is recommended that all tests for determination of friction loss be made by means of a manometer that will give a direct reading of the friction head. This means that the manometer must be connected between the operating penstock and one that is not operating, or else a special pipe must be installed for supplying the static head. Measurements of friction loss made by means of an ordinary pressure gauge are not considered of much value. In case a small pipe is installed for supplying the static head on one leg of the manometer, great care must be used in eliminating errors due to differences in temperature. These tests should preferably be made at night and even then it is found that results are not always reliable. The best possible way to obtain reliable results is to connect the manometer between two penstocks as above outlined. If the pipe is not of uniform diameter, manometers should be connected between the pipe under test and the pipe that is not operating at the points where the diameter changes. These will then give the values of the friction loss for each diameter.

FRICTION COEFFICIENTS

Kind of Pipe	Diameter Inches	Thickness Inches	Value of "N"
1. Welded	72	..	.0112
2. Welded	72	..	.0111
3. Lap and butt riveted	84- 72	3/8-7/16	.016
4. Welded	66- 60	..	.0112
5. Butt riveted	114- 96	3/8-23/32	.017
6. Butt riveted	129	7/16	.0125 .0128
7. Welded	108	1/2-9/16-5/8	.0127 .0131
8. Welded	96	11/16-3/4	.0123 .0127
9. Lap riveted	72	1/4-3/8	.0125
10. Butt riveted	72- 52	3/8-1 1/4	.015

The above tests were made at the following locations:

1. Big Creek No. 8—Southern California Edison Company
2. Big Creek No. 8— "
3. Kern River No. 3— "
4. Kern River No. 3— "
5. Kern River No. 3— "
6. Pit River No. 1—Pacific Gas & Electric Company
7. Pit River No. 1— "
8. Pit River No. 1— "
9. Drum Plant— "
10. Drum Plant— "

Curves showing the results of several of these tests are included.

FRICTION TESTS ON PENSTOCKS OF PACIFIC GAS & ELECTRIC CO.

Location of Penstock	Type of Pipe	Thickness of Pipe	Length Tested Feet	Diameter Inches	Flow C. F. S.	Velocity Ft./Sec.	Derived Coefficients Williams & Hazen's C	Derived Coefficients Kutters N
Drum P. H....	Double lap riveted....	¼" to 5/16"	1489.6	72	70.8	2.50	128	.0125
	“ “ “	“	“	“	86.0	3.04	132	.0120
	“ (10 yrs. old)	“			150–400			.014
	Triple riv. double butt	⅜"	423.9	72	59.9	2.12	105	.015
	“ “ “ “	“	“	“	70.8	2.50	113	.014
	“ “ “ “	“	“	“	86.0	3.04	113	.0139
	“ (10 yrs. old) “	“			150–400			.017
Halsey P. H...	Lap riveted..........	¼" 5/16" & ⅜"	583.4	72	65	2.30	151	.0108
	“ “	“	“	“	70	2.48	135	.0119
	“ “	“	“	“	86	3.04	133	.012
	“ “	“	“	“	104	3.68	128	.0123
	“ “	“	“	“	138	4.88	122	.0127
	“ “	“	“	“	176.5	6.24	116	.0129
	“ “	“	“	“	214	7.57	115	.0129
	“ “	“	“	“	257.5	9.11	114	.0128
	“ “	“	“	“	290	10.26	113	.0128
	“ “	“	“	“	313	11.07	113	.0128
	“ “	“	“	“	332	11.74	114	.0126
	Butt strap...........	⅜"	319.9	“	65	2.30	140	.0118
	“ “	“	“	“	70	2.48	119	.0119
	“ “	“	“	“	86	3.04	120	.0131
	“ “	“	“	“	104	3.68	119	.0131
	“ “	“	“	“	138	4.88	113	.0134
	“ “	“	“	“	176.5	6.24	107	.0139
	“ “	“	“	“	214	7.57	106	.0138
	“ “	“	“	“	257.5	9.11	105	.0137
	“ “	“	“	“	290	10.26	104	.0139
	“ “	“	“	“	313	11.07	104	.0137
	“ “	“	“	“	332	11.74	104	.0137
Wise P. H....	Double lap riveted....	7/16"	744.7	84	130	3.38	110	.01425
	“ “ “	“	“	“	175	4.55	106	.0145
	“ “ “	“	“	“	245	6.37	104	.01435
	“ “ “	“	“	“	320	8.31	104	.0142
	Single lap riveted.....	⅜"	768.0	“	130	3.38	116	.0136
	“ “ “	“	“	“	175	4.55	114	.0136
	“ “ “	“	“	“	245	6.37	110	.01365
	“ “ “	“	“	“	320	8.31	110	.01345
	Butt strap............	⅜" to 7/16"	1070.6	“	130	3.38	116	.0136
	“ “	“	“	“	175	4.55	109	.0138
	“ “	“	“	“	245	6.37	105	.0142
	“ “	“	“	“	320	8.31	105	.0141
Pit No. 1 P. H..	Triple riveted........	7/16"	231.0	129	982	10.81	120	.0125
	Butt joint............	“	“	“	714	7.86	120	.0128
	Lap welded...........	½"-9/16"	515.0	108	982	15.45	114	.0127
	Bump joint..........	and ⅝"	“	“	714	11.22	113	.0131
	“ “	11/16"	240.0	96	982	19.54	115	.0123
	“ “	¾"	“	“	714	14.20	115	.0127

NOTE: Method of Testing used for Drum P.H., Halsey P.H., and Wise P.H.—U-Tube with Carbon Tetrachloride between taps to penstock. Method of Testing used for Pit No. 1-P.H.—U-Tube with Mercury between penstock pipes at upper and lower end of tested section, one penstock being idle. All pipe tested was new excepting those noted.

From the experimental data available it is believed that the following values for "N" are conservative and can be used with safety for purposes of design:

Lap welded pipe with bump joints............	.013
Thin riveted pipe with lap joints..............	.014
Pipe of moderate thickness with butt joints...	.016
Heavy pipe with triple riveted butt joints.....	.018

Air Vents

Standpipes

It is considered necessary to install a standpipe immediately below a valve placed at the upper end of a penstock. This standpipe should be of sufficient size to admit all the air required to prevent the collapse of the pipe under the worst conditions of flow. This case will occur with free discharge at the lower end of the line with the valve at the top closed.

It is generally believed that the air flow through a standpipe or air valve should be calculated on the basis of half the difference of pressure that the pipe can safely withstand without collapse.

In some cases the standpipes have been designed to provide for the maximum flow through the penstock under normal load conditions with the upper valve closed. It is believed, however, that this is not a sufficiently severe case, as the upper valve would be closed in the event of the breaking of

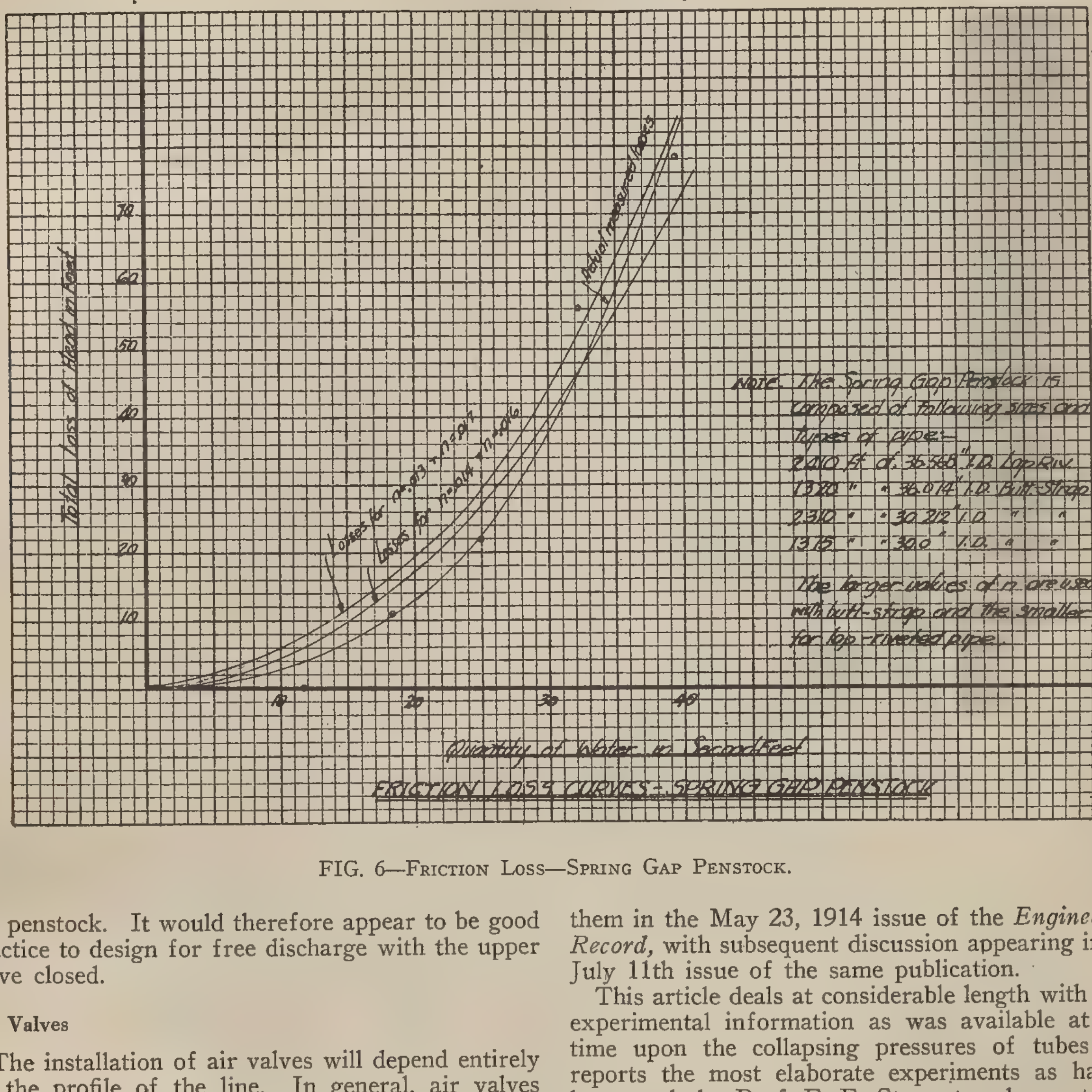


FIG. 6—FRICTION LOSS—SPRING GAP PENSTOCK.

the penstock. It would therefore appear to be good practice to design for free discharge with the upper valve closed.

Air Valves

The installation of air valves will depend entirely on the profile of the line. In general, air valves are located on summits or at abrupt changes of grade where there is a possibility of the water column separating on a suddenly accelerating gradient, or where the hydraulic grade lines closely approach the level of the pipe line.

Air valves are preferably of the vertical check-valve type, constructed of material which will not corrode. Sketches of several satisfactory types are shown herein. Gate valves of the rising stem type are sometimes inserted between the air valve and the pipe.

Protection from the weather should be provided by rainproof shelters, which consist essentially of a shed roof without sides. A very satisfactory way of providing for this is shown in the sketch of the air valve made by the American Spiral Pipe Company.

Some experiments on the collapsing pressure of pipes have been made by M. L. Enger and F. B. Seely, of the Department of Theoretical and Applied Mechanics, University of Illinois, and published by them in the May 23, 1914 issue of the *Engineering Record,* with subsequent discussion appearing in the July 11th issue of the same publication.

This article deals at considerable length with such experimental information as was available at that time upon the collapsing pressures of tubes and reports the most elaborate experiments as having been made by Prof. E. E. Stewart and reported in the transactions of the A.S.M.E. Transactions 1906, Volume 27, page 730. This deals with a series of tests on tubes 23 inches diameter made of Bessemer steel lapwelded. Another series of tests was made during the same year (1906) by Prof. A. P. Carman and L. E. Carr. This is reported in Bulletin No. 5 of the Illinois Experimenting Station. The experiments of Carman and Carr supplement and corroborate those of Prof. Stewart, and the formula, which is practically the same for collapsing pressure on pipes of these sizes, is $P_2 - P_1 = 50{,}200{,}000 \left(\frac{t}{d}\right)^3$ where,

P_2 = outside pressure, pounds per square inch
P_1 = inside pressure, pounds per square inch
t = thickness in inches
d = diameter in inches.

Expansion Joints

With the increasing installation of exposed pipes it becomes more necessary to use some type of ex-

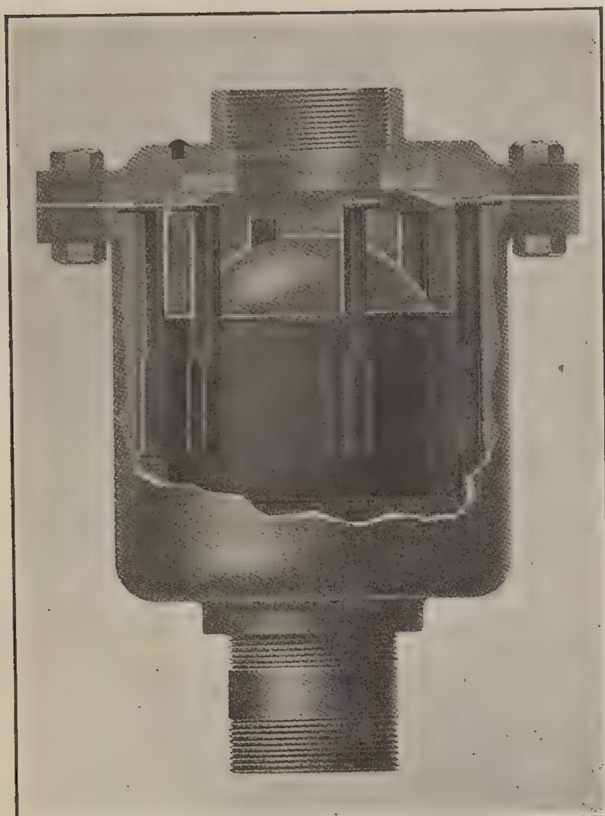


FIG. 7—CRISPIN AIR AND VACUUM VALVE—MULTIPLEX MANUFACTURING COMPANY.

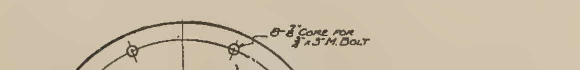


FIG. 8—8-IN. AIR VALVE—CITY OF LOS ANGELES.

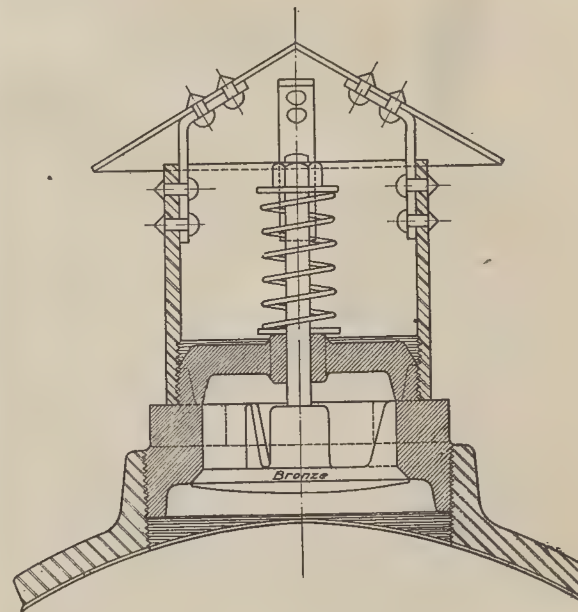


FIG. 9—4-IN. AIR VALVE—AMERICAN SPIRAL PIPE COMPANY.

pansion joint to relieve the pipe of temperature stresses. It is the general opinion that expansion joints must be used with exposed pipes, and there are records of failures of buried pipes that could have been prevented had expansion joints been installed.

There are further advantages in the use of expansion joints; they eliminate excessive longitudinal stress in the pipe and consequently a lighter type of girth joint can be used. They also make the stresses in the entire line at angles, etc., entirely determinate.

A great number of these expansion joints have been used and some of them have been in service for many years. There are no operating objections to these joints, as it is found that, if they are properly packed, the leakage is negligible.

Type

Many types of expansion joints have been devised, but the kind in most general use today is of the simple stuffing box type made up of welded pipe. All of the welded pipe manufacturers are prepared to make these joints, and we are including a sketch of one made by the M. W. Kellogg Company which has the advantage of a copper sleeve on which the packing bears. This is described more fully in the statement by this company at the end of the report.

A sketch is here shown of a simple type of joint that can be made up of riveted pipe.

Location

There is some difference of opinion as to location of expansion joints with reference to the angle points. In many cases the joint has been located midway between anchors largely for the reason that this reduced the movement on the pier to a minimum. There are many cases, however, where this location

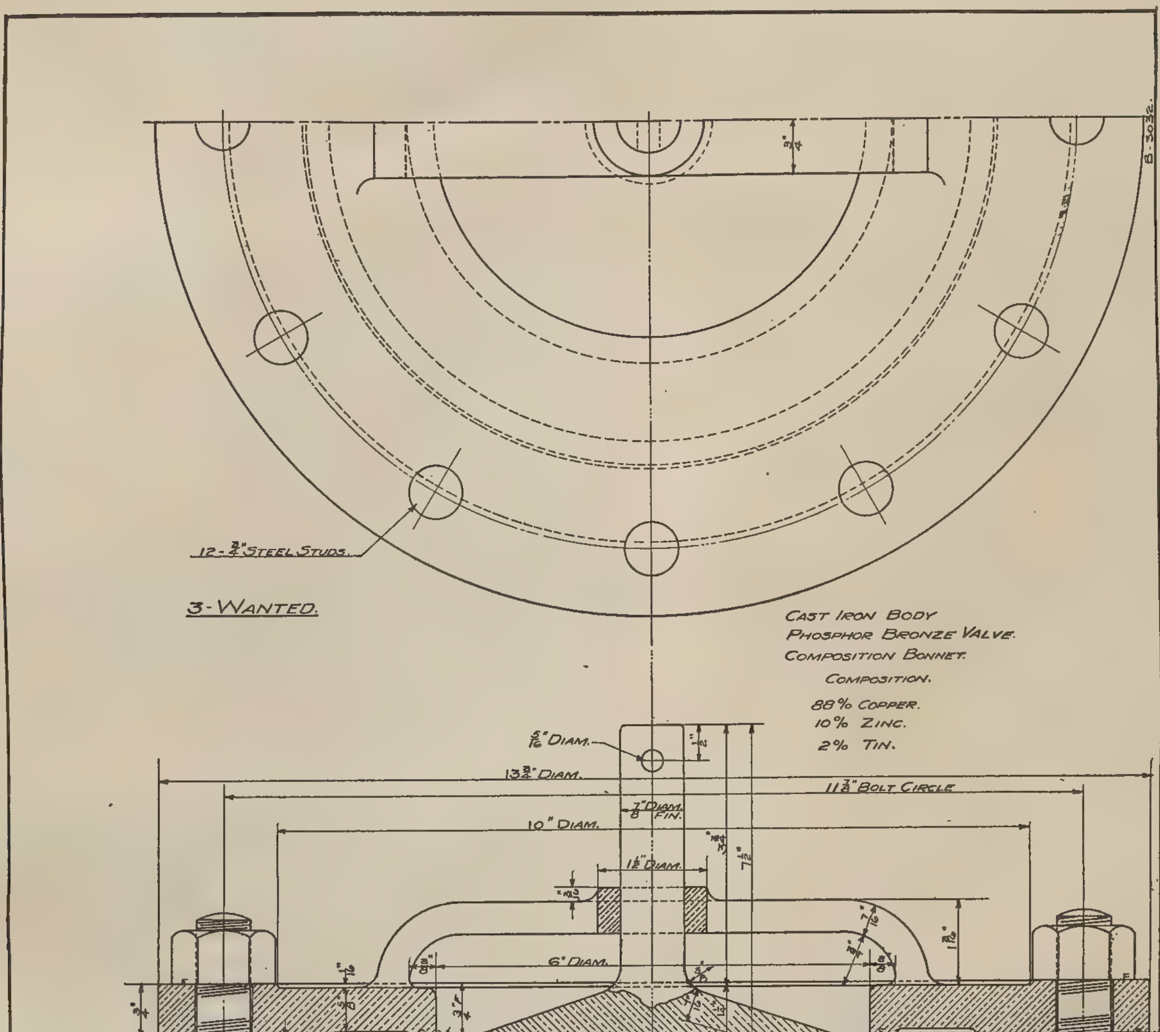


FIG. 10—6-IN. AIR VALVE, DEER CREEK PIPE LINE—CENTRAL CALIFORNIA POWER COMPANY.

might introduce excessive forces on the anchor; and on steep slopes, the installation of the pipe from the anchor down hill would be difficult. The general practice seems to be to install the expansion

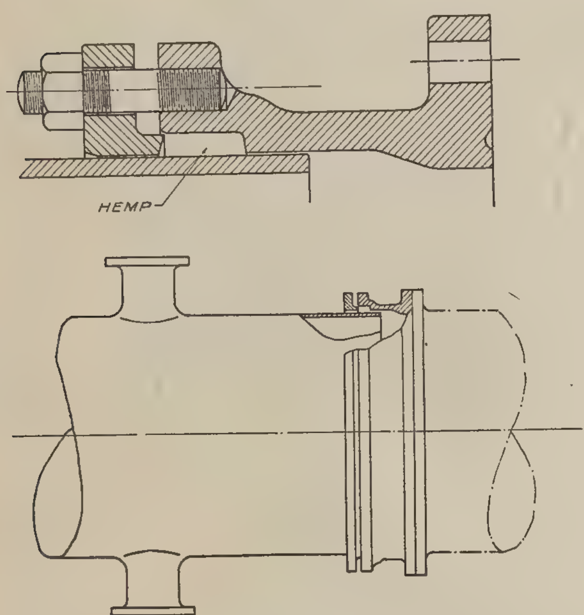


FIG. 11—ERECTION JOINT AT POWER HOUSE—NATIONAL TUBE COMPANY.

joints midway between anchors for flat slopes and directly below the anchor if the slope is steep.

Packing

The packing in most general use is of square braided hemp or flax impregnated with graphite. Very good results have been reported by the use of Crondley centrifugal pump packing; this is a soft cotton packing lubricated with graphite.

The National Tube Company recommend "Gem" Brand Flax Packing made by Speck-Marshall Company of Pittsburgh. They adopted this packing for their hydraulic presses after experimenting with practically all the kinds of packing they could obtain.

Specials

Laterals

Practically the only novel design of lateral available is that developed by the Kellogg Company. A complete description and details of this will be found in their statement at the end of the report.

Bends

It is customary to make bends to radius of 4 or 5 times the diameter of the pipe. It is desirable to have the radius not greater than this, as this short radius allows the entire bend to come within the anchor in most cases. This permits a better distribution of the stress from the pipe to the anchor.

The National Tube Company has suggested that all bends be made with flanged ends. It is claimed that the money saved in testing would more than offset the cost of the flanges. This seems to be a very desirable arrangement, as it is quite difficult to make a pressure test on a bend with bump joints.

The common type of bump joint can be deflected to form an angle of 5 degrees. It is usual shop practice to form welded bends of tangents having a maximum included angle of not over 14 degrees. The American Spiral Pipe Company has developed a novel method of forming a bend by cutting out about two-thirds of the circumference of the pipe and welding together after bending.

Manholes

In general a manhole should be placed at the top and bottom of a line. Intermediate manholes

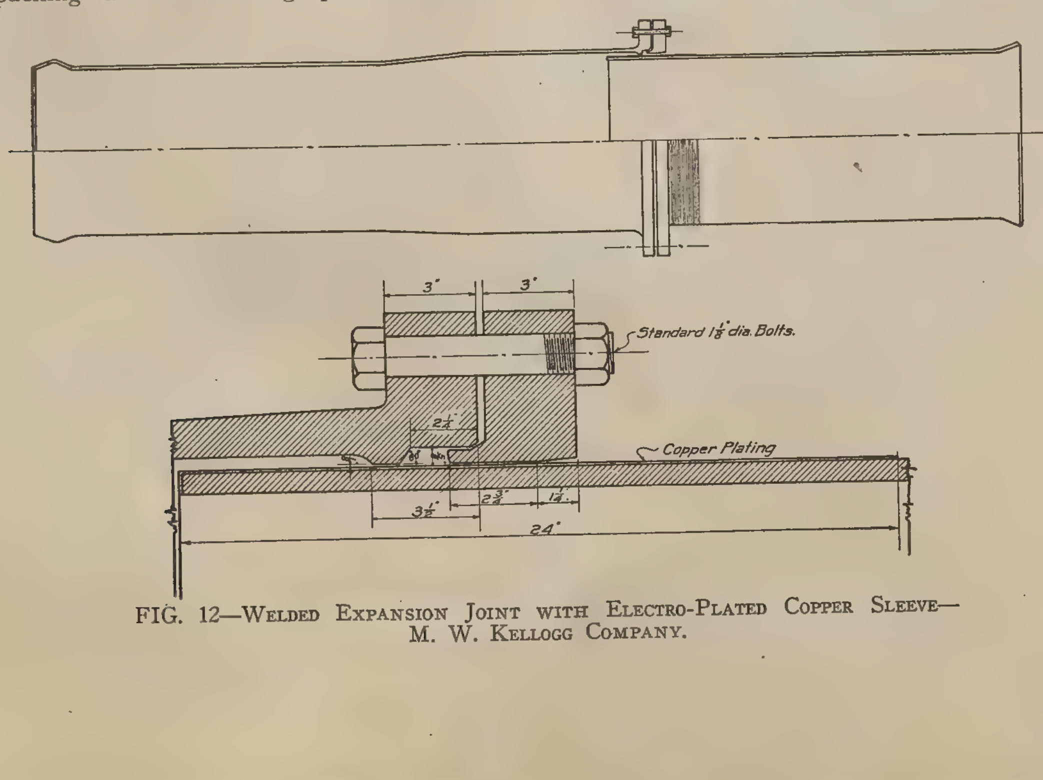


FIG. 12—WELDED EXPANSION JOINT WITH ELECTRO-PLATED COPPER SLEEVE—M. W. KELLOGG COMPANY.

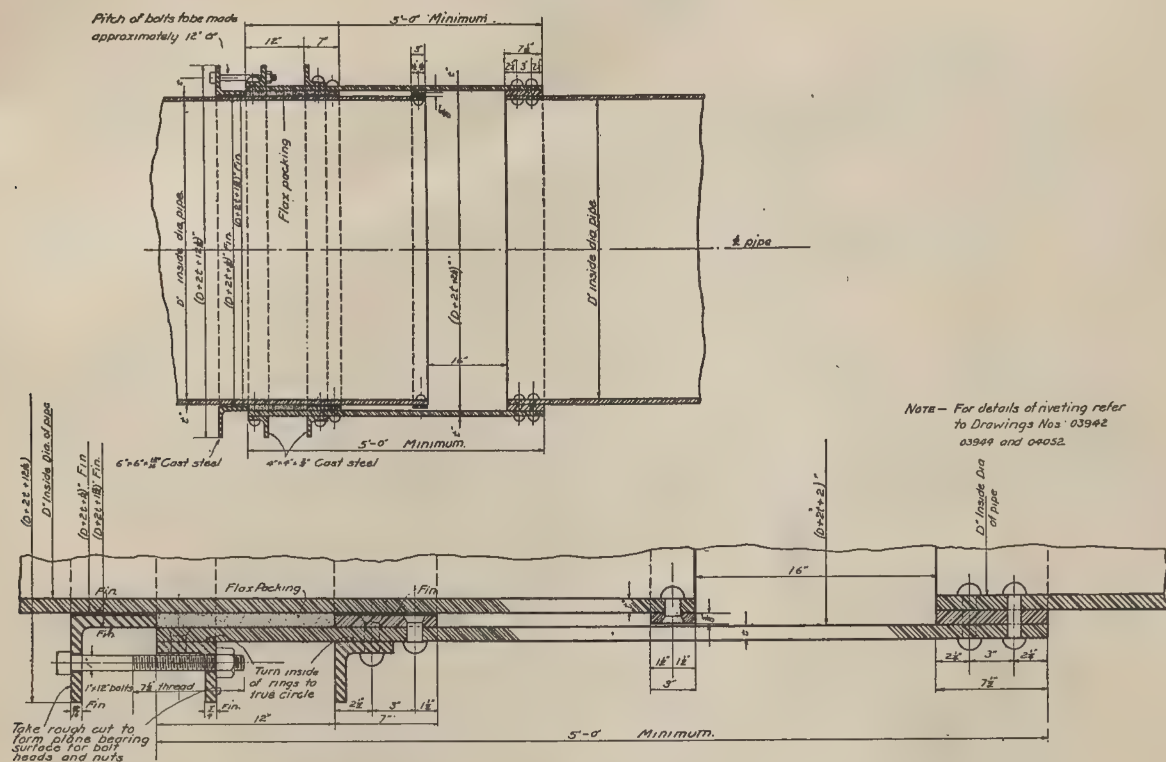


FIG. 13—TYPE OF SLIP JOINT FOR DRUM PIPE LINE, PACIFIC GAS & ELECTRIC COMPANY.

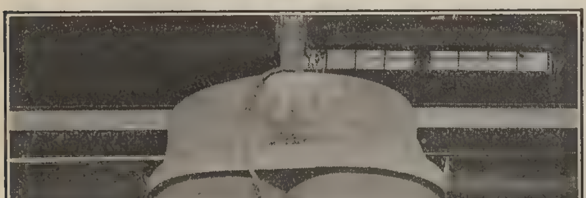


FIG. 14—WELDED LATERAL, 70 IN. BY 48 IN. BY 48 IN.—M. W. KELLOGG COMPANY.

may be used if the line is long. In welded pipe it is customary to use a forged steel manhole frame welded into the pipe; this type produces the minimum disturbance to the flow of water. It is recommended that manholes not less than 16 or 18 inches in diameter be used.

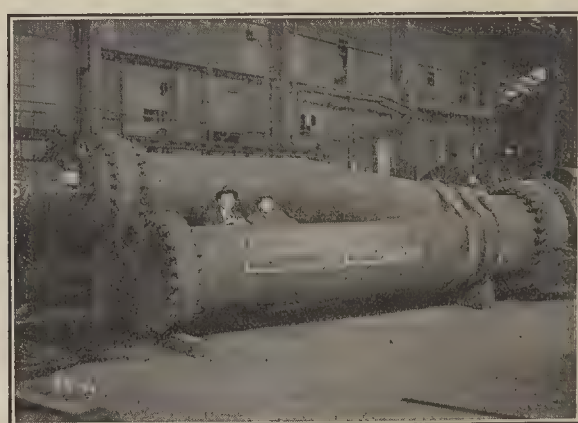
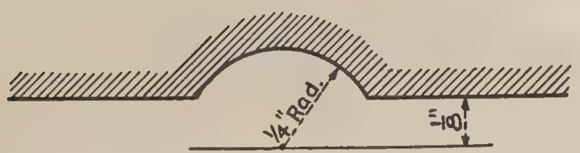


FIG. 15—WELDED LATERAL, 46 IN. BY 36 IN. BY 36 IN., 1 1/16 IN. PLATE, 28,000 LB., 600 LB. TEST PRESSURE—M. W. KELLOGG COMPANY.

Gaskets

For practically all pressures encountered in penstock work, it has been found that a round gutta percha gasket is the most satisfactory to use. A 1/4-inch round gasket with ends scarfed and cemented, placed in a groove as shown in the sketch has given perfect service for heads up to 2,100 feet.



In using this gasket, only the face of one flange is grooved, the other is plain. The gasket is held in the groove by means of rubber cement.

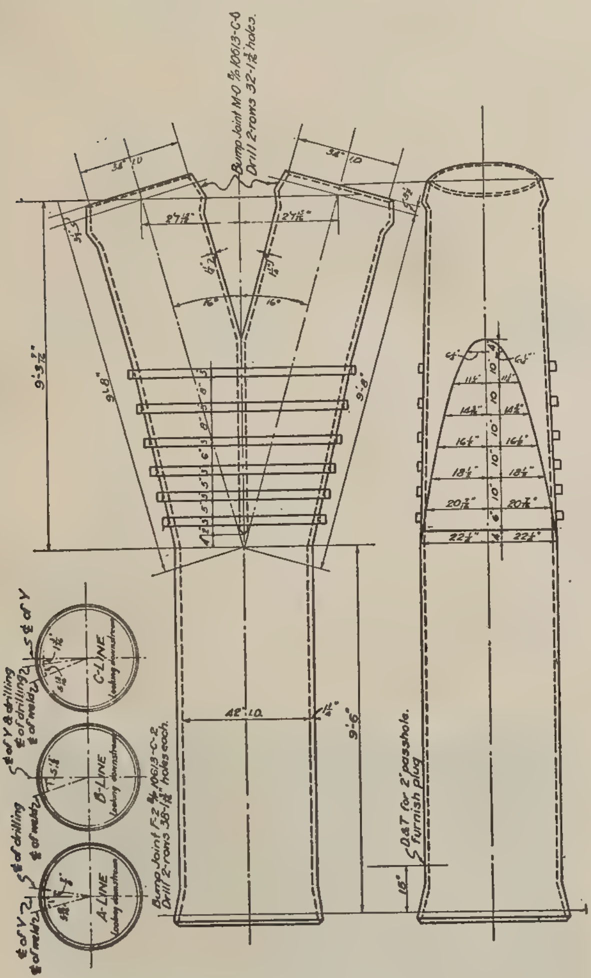


FIG. 16—DETAIL OF LATERAL, 34 IN. BY 34 IN. BY 42 IN. —M. W. KELLOGG COMPANY.

Flanges

There is no standard as to flange details; the penstock flange seems, in general, to be made to suit the flange of the water wheel or valve to which it is bolted.

Inaccessible bolts should be avoided. For close bolt spacing, alternate bolts should have washers to offset the heads so a wrench can be applied. Flanges that are slipped on to the pipe should have the neck bored. The back face of all flanges should be spot faced or rough turned.

A sketch is included showing a type of loose flange used by the City of Los Angeles to permit the removal of a needle valve from the line.

Nozzles, etc.

In general, it is customary to use welded nozzles and other fittings on welded pipe, although one manufacturer believes that for thickness of pipe wall over 1¼ inch steel castings should be used, riveted to the pipe.

Saddle castings should be ground to fit the curve of the pipe and a caulking strip used whether the castings are of cast steel or cast iron.

Anchor Rings

The function of an anchor is to exert on the pipe a force equivalent to the resultant of all the forces acting at the angle in the pipe. If the anchor can be designed so as to properly supply this resultant force, it is seen that anchor rings on the pipe are unnecessary because there can be no tendency for the pipe to move in the anchor.

In some cases, at an angle, it may not be possible to locate the anchor so that it will take the place of the resultant, and then anchor rings should be used. If it is ever necessary to install an anchor in a straight run of pipe, anchor rings or their equivalent must be used to transmit the force from the pipe to the anchor.

Anchor rings of heavy rectangular section have been used on welded pipe, the rings being acetylene welded to the pipe. For riveted pipe anchor rings are, of course, riveted to the pipe. Welded rings on welded pipe are preferred as it is desirable to eliminate rivets encased in concrete wherever possible. Anchor rings to be riveted to the pipe are best made of a channel with the back to the pipe. This is preferred to an angle, as it is not possible for the outstanding leg of an angle to take much load at its outer end.

Anchors

Design

The general practice is to so design anchors that they will properly take care of all resultant forces on the bend by gravity only. Local conditions rarely permit the use of anchor rods as this would require very solid rock to insure safety. Where anchor bolts can be used, the skeleton type of anchor designed by the Kellogg Company would be very satisfactory. This can be used either with anchor bolts grouted into solid rock or attached to a concrete base. A sketch of this skeleton anchor is included in this report.

In general, anchors are placed only at angle points. If a line is made up of very long tangents, anchors would probably be needed between angles to equalize the movement of the pipe on the piers, but in ordinary installations these are not needed.

Wherever possible, anchors should be designed so that no riveted joints will be enclosed in concrete. This, of course, cannot be done with riveted pipe, but is generally possible with welded pipe. Sometimes it may be necessary to lengthen the legs of the bend in order to accomplish this result.

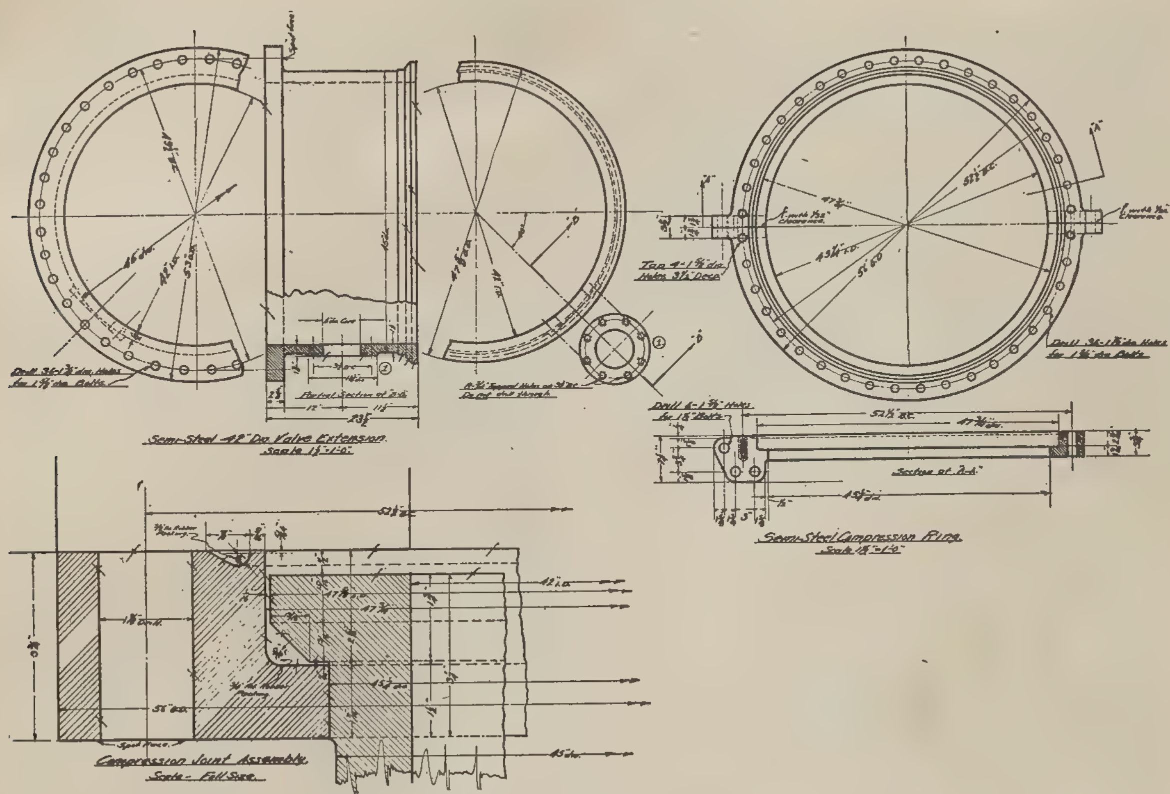


FIG. 17—DETAIL OF LOOSE FLANGE TO PERMIT REMOVAL OF PENSTOCK VALVE, CITY OF LOS ANGELES.

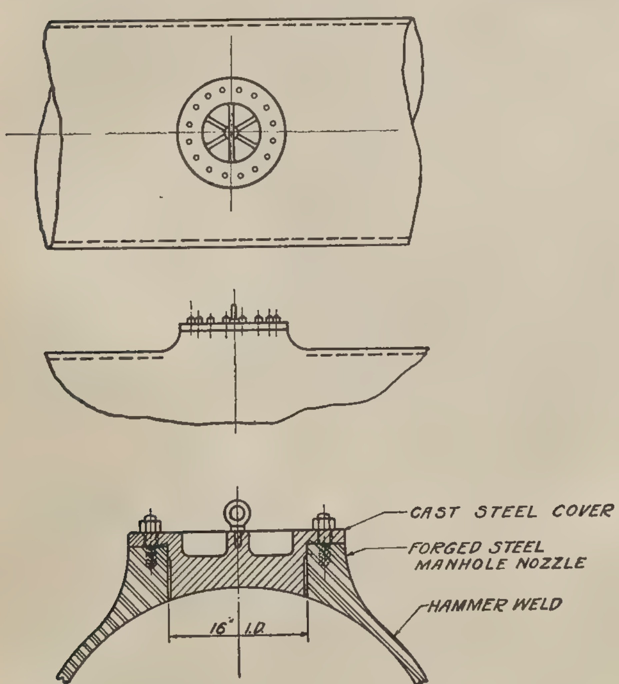


FIG. 18—WELDED FORGED STEEL MANHOLE—M. W. KELLOGG COMPANY.

The base of the anchor should be kept away from the pipe at least 18 inches to permit caulking of the riveted joints. The base is always placed first and the cap surrounding the base poured after the pipe has been installed.

Analysis of Stresses

The following formulae and diagrams give a complete analysis of all of the stresses acting upon an anchor. Many of these forces will generally be of small magnitude, but they have been included in this discussion in order to make it complete. As shown on the diagram, all of these forces acting at the bend are finally combined into vertical and horizontal components, which in turn, combined with the weight of the anchor itself, give a resultant that must lie within the middle third of the anchor base.

Definition of Symbols

h = static head at any point
A = inside area of pipe in square inches
p = pressure per square inch at any point
a = angle of upstream leg with horizontal
b = angle of downstream leg with horizontal
c = exterior horizontal angle between center lines of upstream and downstream legs
d = true deflection angle of bend
W = weight of pipe from a point midway between anchor and first pier, upstream to expansion joint
W′ = weight of pipe from a point midway between anchor and first pier, downstream to expansion joint
P = weight of water in pipe W
P′ = weight of water in pipe W′
Q = weight of water and pipe between points midway between adjacent piers above and below anchor
f = friction coefficient of pipe on piers
V = velocity of water in feet per second

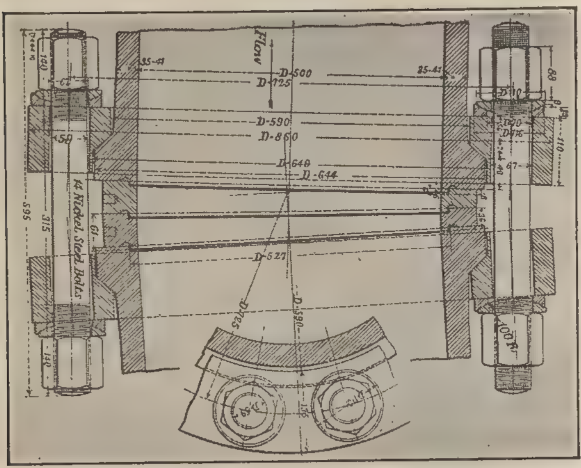


FIG. 19—ADJUSTABLE HIGH PRESSURE JOINT USED AT 5000 FT. HEAT PLANT, AT FULLY, SWITZERLAND.

C = circumference of pipe in feet at expansion joint above anchor
C' = circumference of pipe in feet at expansion joint below anchor
S = weight of pipe from center line of anchor to point midway between adjacent upstream pier
S' = weight of pipe from center line of anchor to point midway between adjacent downstream pier
F = friction in expansion joint per lineal foot
E = area of exposed end of pipe at expansion joint in square inches
R = area of pipe in square feet
w = weight of water per cubic foot
g = acceleration of gravity
K = reduction in area at reducer above anchor
K' = reduction in area at reducer below anchor

Penstock Anchor Forces

1. Friction on supports due to expansion or contraction on upstream side of anchor $= f\ (W + P)\ \cos.a.$
2. Friction on supports due to expansion or contraction on downstream side of anchor $= f\ (W' + P')\ \cos.b.$
3. Friction of expansion joint on upstream side of anchor due to expansion or contraction $= CF$ ($F = 500$ lb. per lineal foot determined by experiment).
4. Friction of expansion joint on downstream side of anchor due to expansion or contraction $= C'F$.
5. Force due to deadweight of pipe acting parallel to center line of pipe on upstream side of anchor $= (W + S)\ \sin.a.$
6. Force due to deadweight of pipe acting parallel to center line of pipe on downstream side of anchor $= (W' + S')\ \sin.b.$
7. Force due to bend in pipe with water flowing $= RVwV \div g$.
8. Hydrostatic force at anchor $= pA$ (acts along axis of pipe toward bend on each side of anchor).
9. Force on exposed end of expansion joint upstream side of anchor $= Ep$.
10. Force on exposed end of expansion joint downstream side of anchor $= E'p$.
11. Force due to reducer above anchor $= pK$.
12. Force due to reducer below anchor $= pK'$.
13. Horizontal component in plane of upper penstock center line for algebraic sum of all forces on anchor, expanding condition.
14. Horizontal component normal to plane of upper penstock center line for algebraic sum of all forces on anchor, expanding condition.
15. Vertical component for algebraic sum of all forces on anchor, expanding condition.
16. Horizontal component in plane of upper penstock center line for algebraic sum of all forces on anchor, contracting condition.
17. Horizontal component normal to plane of upper penstock center line for algebraic sum of all forces on anchor, contracting condition.
18. Vertical component for algebraic sum of all forces on anchor, contracting condition.

Piers

General Design

Piers are made a minimum width of three feet in the direction of the pipe and have a bearing on the pipe of 60 deg. to 120 deg. It has been found that the corners of the 120 deg. piers invariably crack, and therefore there is a decided tendency to reduce the arc of contact to a smaller amount. The pier must be designed to resist overturning due to the frictional force between the pier and pipe.

In nearly all important installations piers have been spaced about 20 feet apart, or one to each length of pipe. These piers are placed about 4 feet below the upper end of the pipe. In some recent installations of heavy pipe the piers are spaced 40 feet apart when calculation shows that no excessive stresses are caused thereby. When piers are spaced 40 feet they are located in the center of a length of pipe so the girth joints come at the quarter points of the span. The pipe then acts as a continuous girder with the joints at the points of contra-flexure taking practically no stress due to bending.

A sketch of a typical concrete pier is included, also one showing a steel support to be used where the pipe is some distance from the ground. It is best practice to install the pipe on temporary supports and pour the permanent concrete piers after the pipe has been filled with water.

Coefficient of Friction of Pipe on Piers

The method of supporting the pipe on the pier has not received a great amount of consideration. In the majority of cases the pipe rests directly on the concrete with sometimes a layer of roofing paper between. There are also installations where the pipe rests on a cast iron saddle; a sketch of this is included.

In order to reduce the friction on the piers, the M. W. Kellogg Co., have developed a "Roller Bearing Support" and there is included herein a report of tests to determine friction, also a drawing and photographs.

In the design of anchors, one of the principal

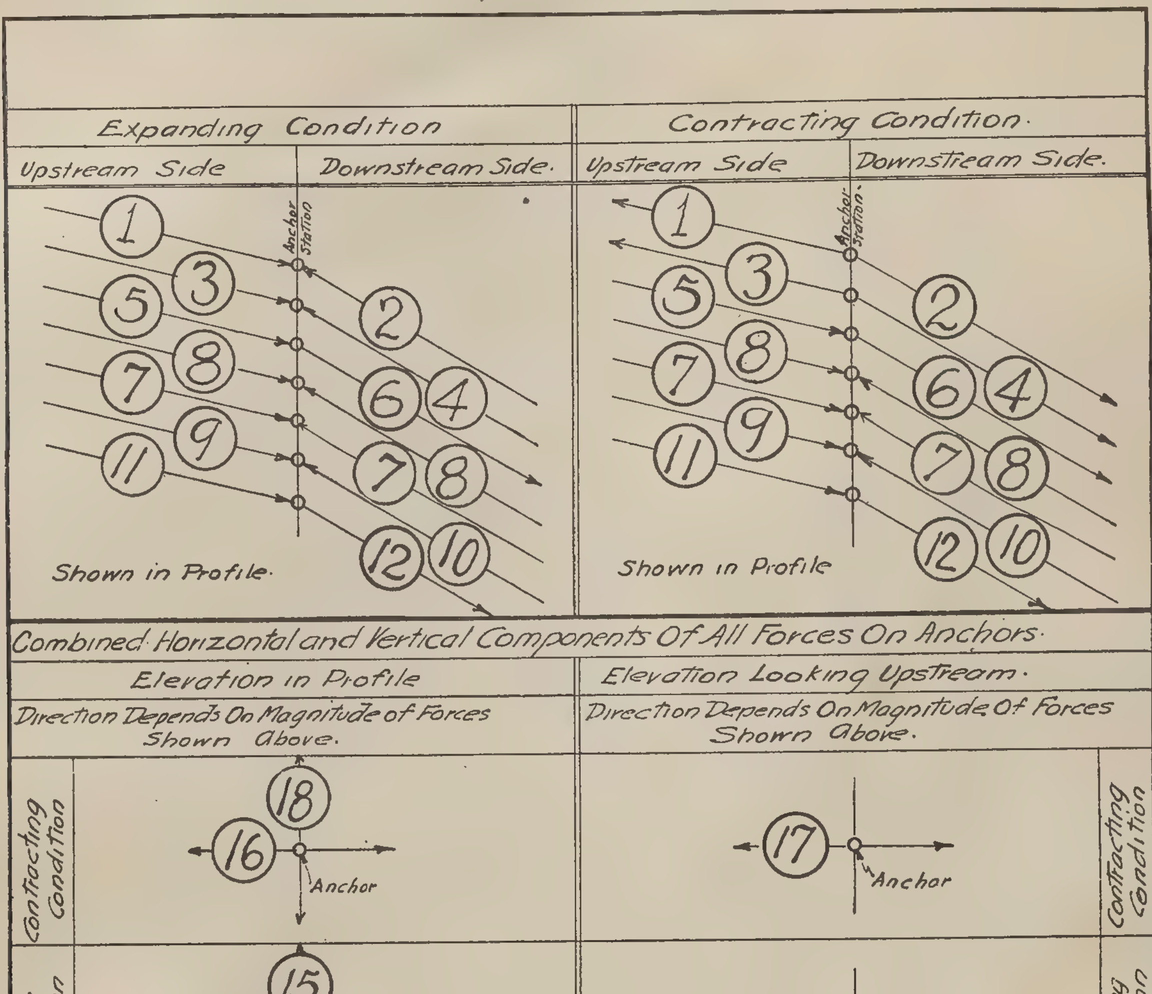


FIG. 20—DIAGRAMS SHOWING EXTERIOR FORCES ON ANCHOR.

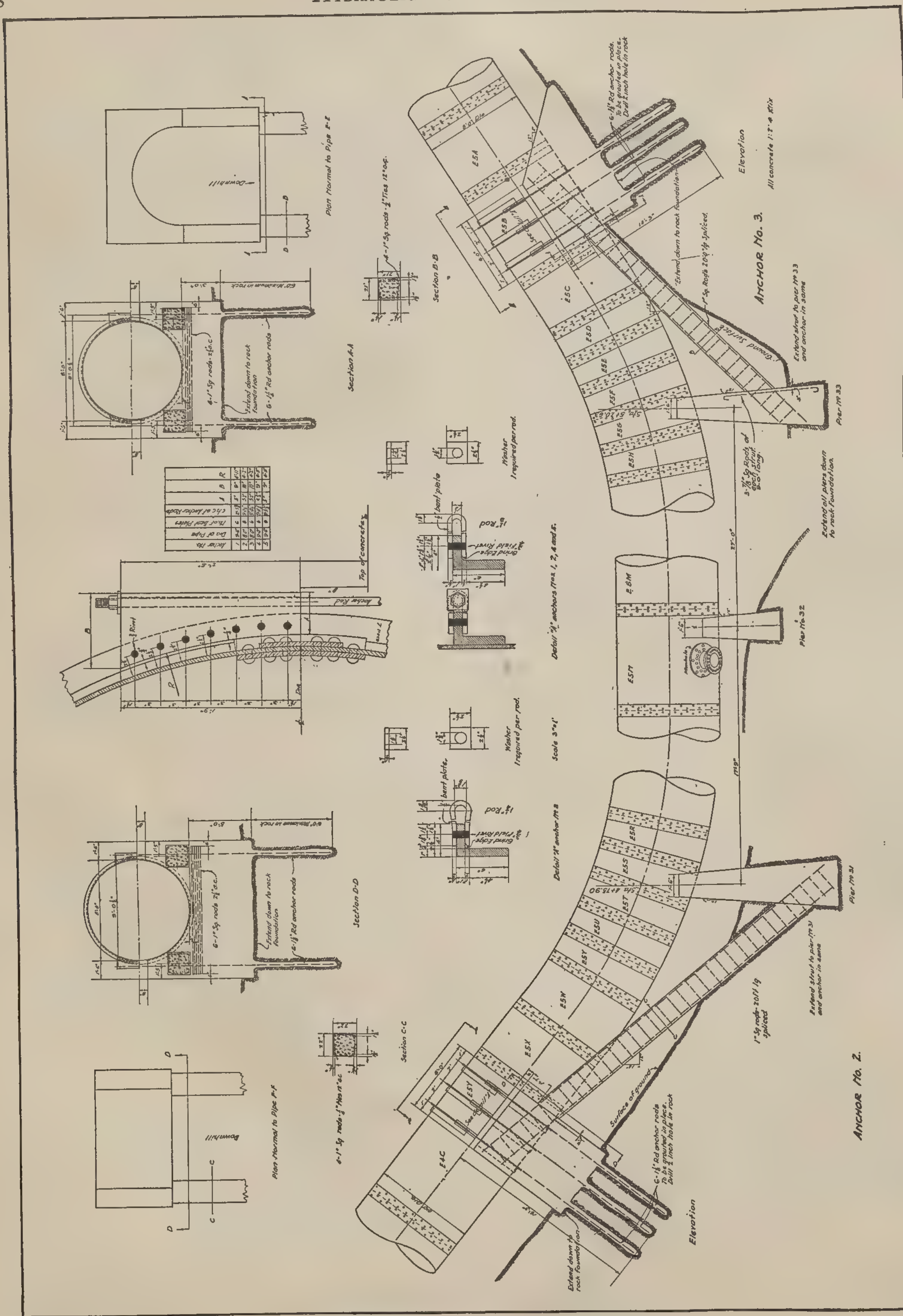


FIG. 21—DETAIL OF ANCHORS, KERN RIVER NO. 3 SIPHON—SOUTHERN CALIFORNIA EDISON COMPANY.

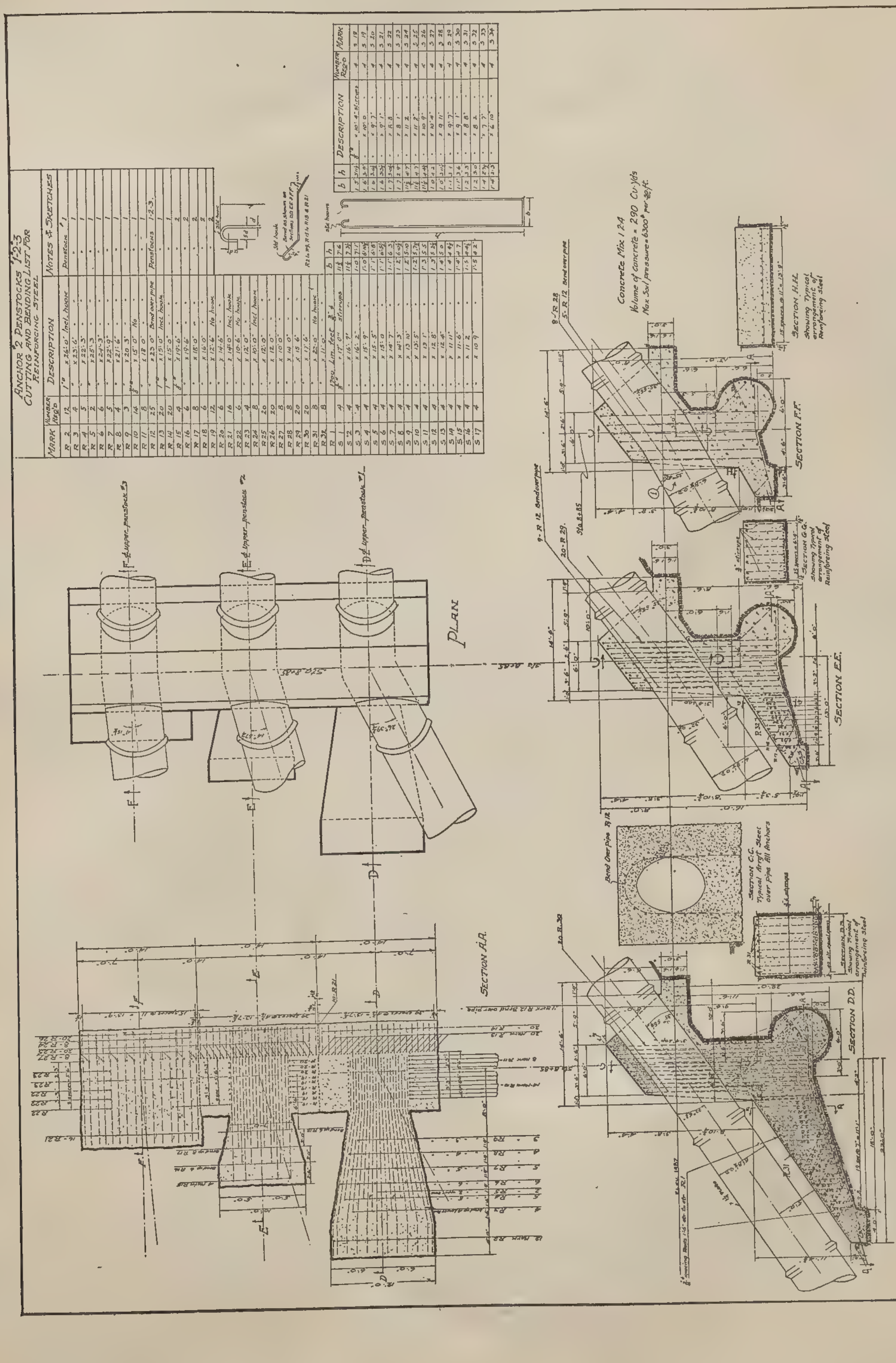


FIG. 22—DETAIL OF ANCHORS, BIG CREEK NO. 3—SOUTHERN CALIFORNIA EDISON COMPANY.

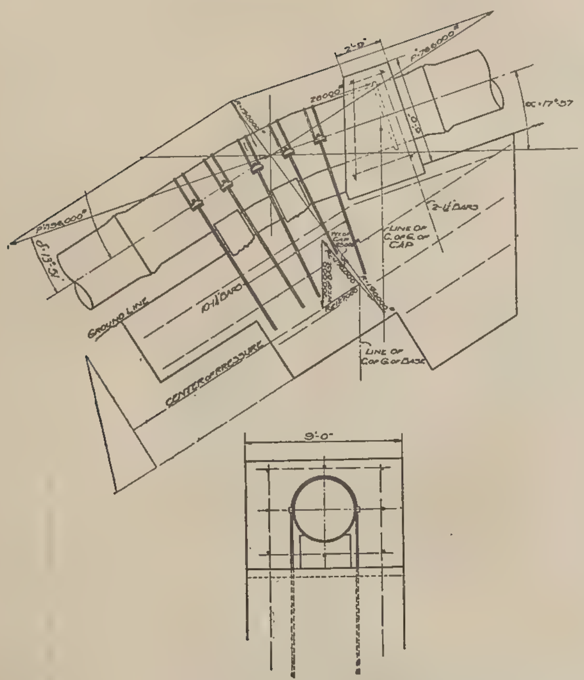


FIG. 23—SKELETON ANCHOR WITH BASE — M. W. KELLOGG COMPANY.

forces is that caused by friction on piers during expansion or contraction of the pipe. With a view to finding some combination other than a roller bearing which would show a minimum frictional coefficient, the Southern California Edison Company has completed a series of tests, attempting by oscillations to approximate field conditions of movement of the pipe.

The tests were all made with pressure applied to a steel plate bearing on either concrete or cast iron with materials between them as indicated in the following table of test data. The tests from No. 7 to No. 24 inclusive were from rest with the surfaces in their original condition, and hence are not as representative of actual conditions as the other tests that were made after the surfaces were worn together by the reciprocating motion.

Test No. 1

Two pieces 4 x 4 x 1/16 in. thick Johns-Manville Company service sheet packing, one fastened to concrete and the other to a steel plate with a thin layer of graphite grease between faces of packing. A load of 110 lb. per square inch was applied and an oscillating motion with a stroke of one inch in length at the rate of 1¾ strokes per minute was set up. The coefficient at the start was determined and motion continued for 500 strokes. Motion was stopped for 15½ hours and then resumed until 1,000 strokes had been registered.

Test No. 2

Same as No. 1, without grease. Ten hours in motion for 500 strokes, 30 hours at rest under load. Two days later coefficient of friction at start=.28; coefficient of friction at 500 strokes=0.33.

Test No. 3

One sheet thin brass and one piece 4 x 4 x 1/16 in. Johns-Manville Company service sheet packing without grease; 9½ hours in motion; 16 hours at rest under load; otherwise the same as No. 1.

Test No. 4

One-quarter in. steel plate sliding over a 4 x 4 in. concrete surface. Motion 1.45 strokes per minute; load 110 pounds per square inch; total wear on concrete surface was 1/16 in. deep.

Test No. 5

One-quarter in. steel plate sliding over a 4 x 5 in.

TABLE OF TEST DATA ON COEFFICIENT OF FRICTION

Test No.	Start	200 Strokes	400 Strokes	500 Strokes	600 Strokes	800 Strokes	1000 Strokes
1	.065	...	...	.204	...	...	.270
2	.230	...	...	.400	...	...	.440
3	.180	...	...	.250	...	...	.300
4	.600	.860	.830	...	.790	.720	.700
5	.250	.480	.530	...	.550	.550	.550
6	...	Paper crumpled after 50 strokes					
7	.643	Steel on concrete—pressure 15 lb. per square inch					
8	.667	Steel on concrete—pressure 30 lb. per square inch					
9	.557	Steel on concrete—pressure 45 lb. per square inch					
10	.505	Steel on concrete—pressure 60 lb. per square inch					
11	.56	Steel on concrete—pressure 75 lb. per square inch					
12	.49	Steel on concrete—3-ply roofing paper between pressure 30 lb. per square inch					
13	.445	Steel on concrete—3-ply roofing paper between pressure 60 lb. per square inch					
14	.493	Steel on steel—rusty plates—pressure 15 lb. per square inch					
15	.663	Steel on steel—rusty plates—pressure 30 lb. per square inch					
16	.517	Steel on steel—rusty plates—pressure 45 lb. per square inch					
17	.585	Steel on steel—rusty plates—pressure 60 lb. per square inch					
18	.477	Steel on steel—rusty plates—pressure 75 lb. per square inch					
19	.260	Steel on steel—greased plates—pressure 15 lb. per square inch					
20	.243	Steel on steel—greased plates—pressure 30 lb. per square inch					
21	.237	Steel on steel—greased plates—pressure 45 lb. per square inch					
22	.260	Steel on steel—greased plates—pressure 60 lb. per square inch					
23	.230	Steel on steel—greased plates—pressure 75 lb. per square inch					
24	.793	Steel on concrete bonded					

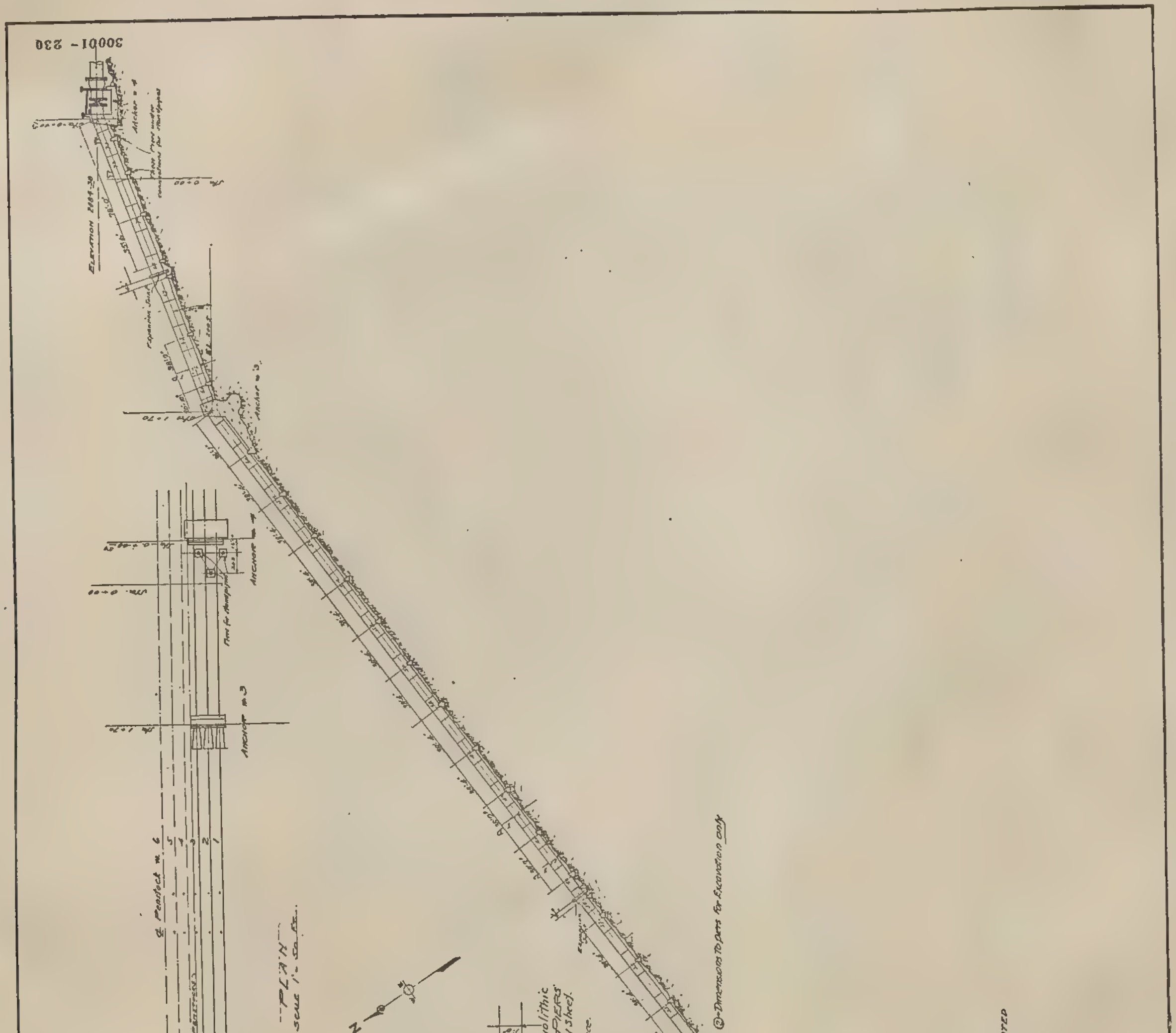


FIG. 24—LAYOUT OF PENSTOCKS, BIG CREEK NO. 3—SOUTHERN CALIFORNIA EDISON COMPANY.

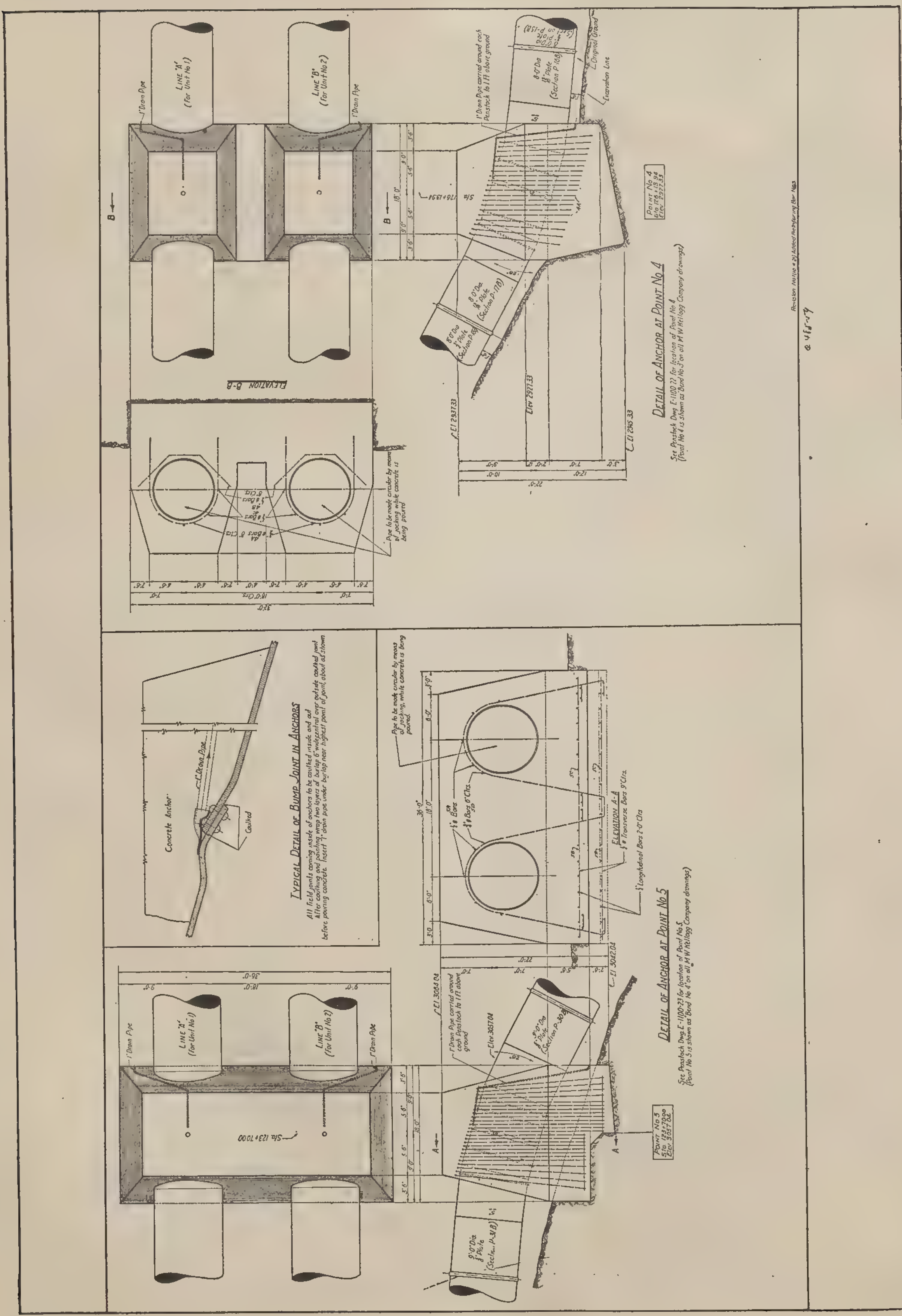


FIG. 25—PENSTOCK ANCHORS, PIT RIVER DEVELOPMENT NO. 1—PACIFIC GAS & ELECTRIC COMPANY.

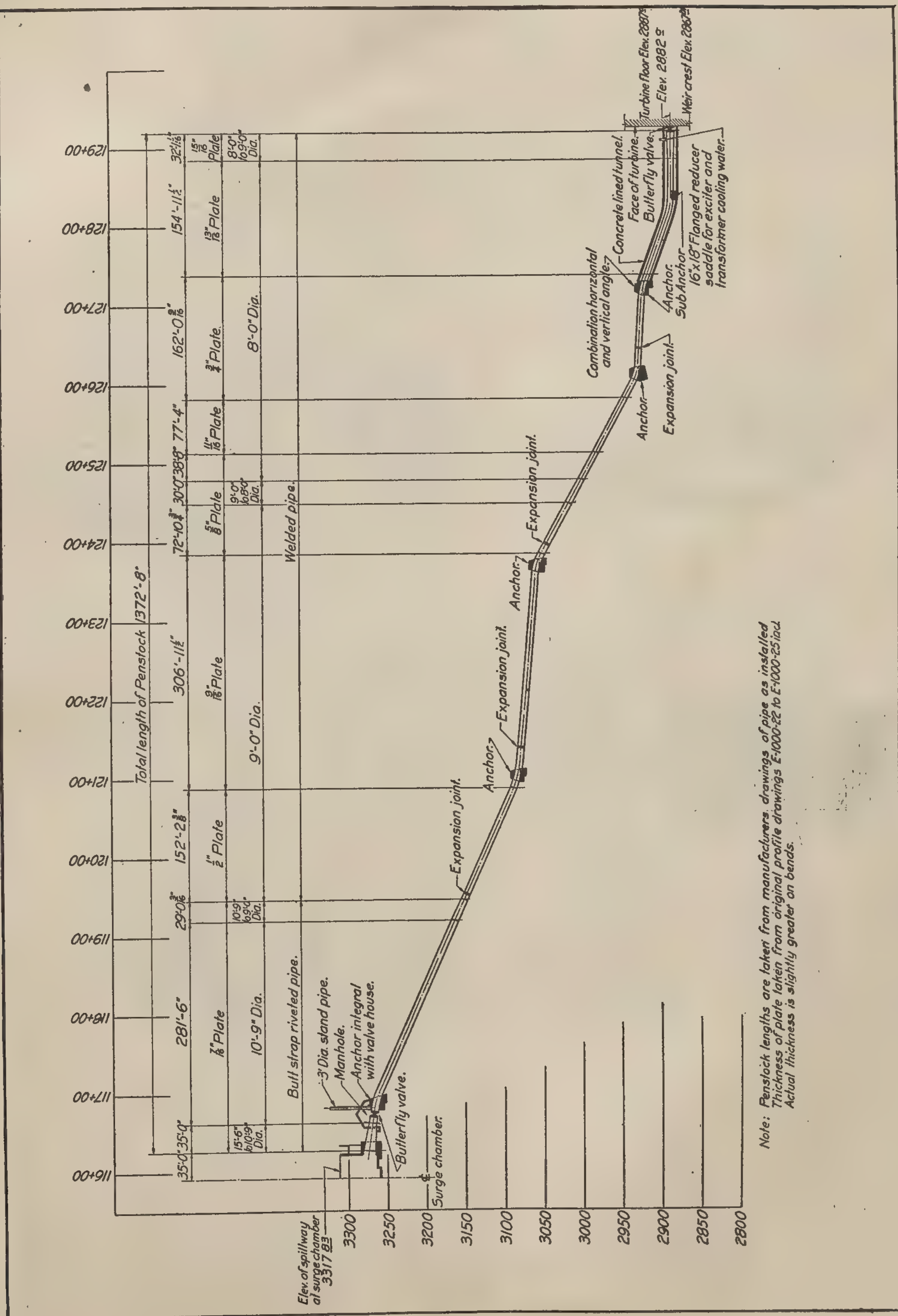


FIG. 26—DIAGRAMMATIC PROFILE OF PENSTOCK, PIT RIVER DEVELOPMENT No. 1—PACIFIC GAS & ELECTRIC COMPANY.

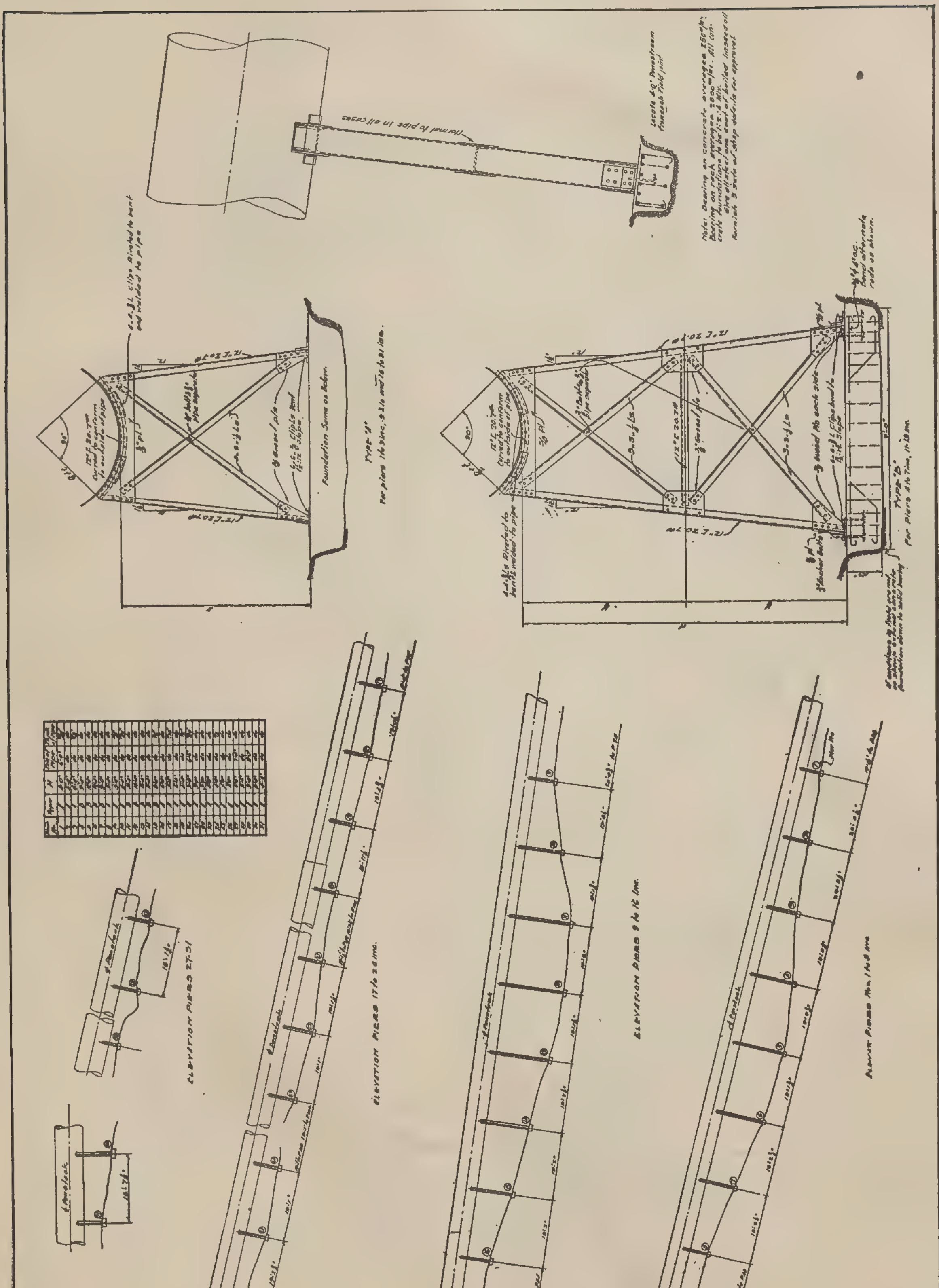


FIG. 27—DETAIL OF STEEL SUPPORTING PIER—SOUTHERN CALIFORNIA EDISON COMPANY.

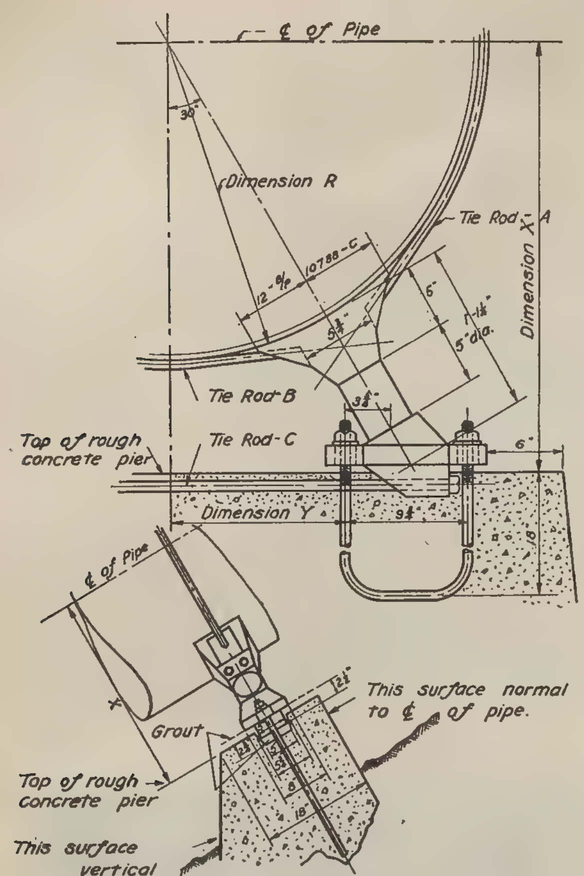


FIG. 28—DETAIL OF ROLLER BEARING SUPPORT—M. W. KELLOGG COMPANY.

cast iron surface; load 110 pounds per square inch; stroke one inch; motion 1¾ strokes per minute; slight amount of heat was noticeable at all times.

Test No. 6

Steel on concrete with asphalt roofing paper between; load 110 lb. per square inch; motion 1¾ per minute; stroke one inch.

From the above tests it would appear that the use of two layers of service sheet packing coated between layers with graphite grease furnishes the lowest frictional coefficient and also has good wearing value. The top sheet should be fastened in some manner to the pipe to guard against collection of moisture between the pipe and packing and also to prevent slipping.

It does not appear to be safe to design for a coefficient less than 0.50 as the deterioration of the surfaces will cause an increase in friction.

Tests for Coefficient of Friction of Roller Bearing Supports

Three pipe sections were bolted together and the open ends closed with plates electrically welded to the pipes. The pipes were 42 in. I.D. x ⅜ in. thick by 19 ft. 6 in. long, each. After filling them with water they were placed horizontally on the roller bearing pier supports and a force was applied at one end by a hydraulic piston until the pipes moved.

Case 1. In which one set of supports was used under the center of each pipe section:

DIRECTION FORWARD Area of piston, 38.5 sq. in. Pressure Gauge Readings	DIRECTION BACK Area of Piston, 31.4 sq. in. Pressure Gauge Readings
10	40
30	50
65	80
10	40
60	50
60	80
20	40
45	60
55	70
10	40
30	60
55	70
10	45
30	45
60	70
(15) 6 545	(15) 850
36.3 × 38.5 = 1400 lb.	56.6 × 31.4 = 1780 lb.
	1400
	2) 3180
	1590 lb. avg. force to move

Weight of pipe (estimated)	12,000 lb.
Weight of water (estimated)	35,000 lb.
	47,000 lb.

Coefficient of friction .... $\frac{1,590}{47,000} = 3.38\%$

Case 2. In which one set of roller bearing pier supports was used under the center of every other pipe section.

DIRECTION FORWARD Area of piston—38.5 sq. in. Pressure Gauge Readings
140
140
120
100
100
5) 600
120 × 38.5 = 4,620 lb.

In this case weights were hung on the ends of the pipes to balance the weight of the intermediate section.

Weight of pipe (estimated)	12000 lb.
Weight of water (estimated)	35000 lb.
Counterweights (actual)	20000 lb.
	67000

Coefficient of friction .... $\frac{4620}{67000} = 6.9\%$

In these tests the bearings were placed on timber cribbing. The more or less erratic results may there-

FIG. 29—ROLLER BEARING SUPPORT—M. W. KELLOGG COMPANY.

fore be attributed to the crushing of these timbers allowing the case castings to tilt. This could not occur if the castings were solidly grouted in concrete. Owing to the fact that the pipe used was awaiting shipment to India this could not be done.

During Case 2 tests the timbers were crushed so badly due to the increased load that it was necessary to replace them with steel plates.

In all the tests the pipes were level on top but there was an uphill effect due to a slight misalignment of the base castings. In the second case the pipe always rolled back to its starting position as soon as the pressure was removed.

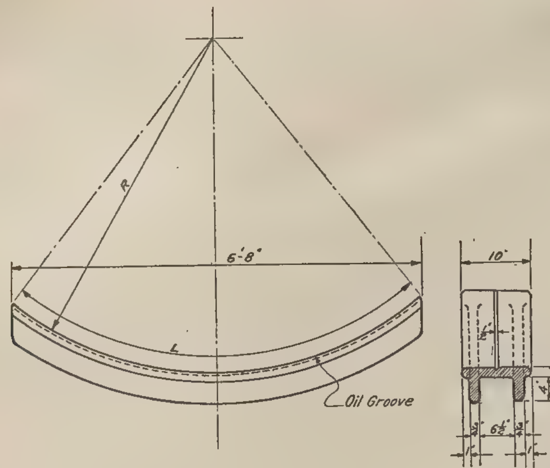
Difficulty was also found in determining the exact pressure at which the pipe began to move. For example, at 140 lb. it took 20 minutes to move the pipes 2½ in. and at 180 lb. the increment of time was only slightly larger. Undoubtedly the pipe began to move at less than 100 lb. in the second case, but without an electric contact indicator and a method of applying a steady pressure, the motion could not be accurately read.

Toward the end of the tests the pipes moved more easily, indicating that the castings were wearing smooth.

The castings were used as they came from the foundry with no machining or smoothing.

In the first case using the maximum pressure reading the factor is only 5.3 per cent while in the second case the average is 6.9 per cent so that a factor of 10 per cent seems more than justifiable for these bearings. The errors in alignment would tend to balance themselves on a long line and the actual

FIG. 30—CONNECTION BETWEEN WOOD STAVE AND STEEL PIPE, 6 FT. DIAMETER, 300 FT. HEAD.



№ REQ'D.	RADIUS R	LENGTH L	PATTERN №		APP. WEIGHT
52	4'-1 13/16"	7'-8 3/4"		SF 1	385#
70	4'-0 3/8"	7'-10 9/16"		SF 2	393#
82	3'-10 15/16"	7'-11 3/4"		SF 3	399#

FIG. 31—CAST IRON SUPPORTING SADDLE—CITY OF LOS ANGELES.

friction factor would undoubtedly be about 5 per cent.

This type of support is very easily assembled in the field and may be adjusted on the pipe.

We believe that a spacing of 40 ft. between these supports is close enough for most conditions in the field.

Reinforced Concrete Pipe

Very little information is available on concrete pipe installation. Probably one of the most important installations is in connection with the Los Angeles Aqueduct. The following description of these pipes is taken from the "Complete Report and Construction of the Los Angeles Aqueduct," published in 1916 by the Los Angeles Department of Public Service.

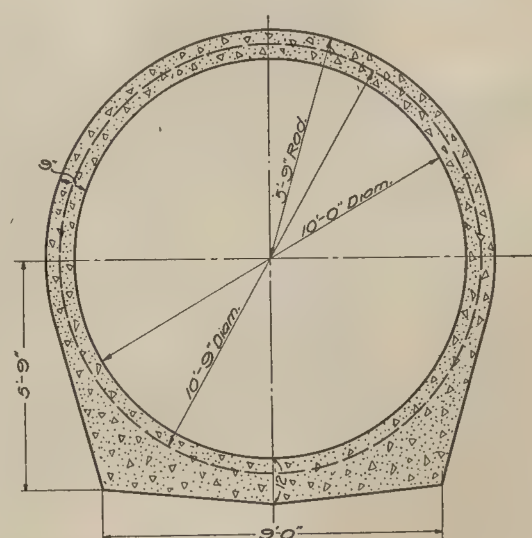
Concrete Pipe

There are eleven concrete pipes on the Los Angeles Aqueduct, all of which are 10 feet in diameter with a 9 in. wall, and with heads ranging from 40 up to 75 feet. The reason for building concrete pipes, was that they were estimated to be cheaper and better for heads less than 75 feet. This is approximately the pressure under which water will begin seeping rather freely through rich concrete. Quarter-inch steel plate for a 10-foot pipe, with the factor of safety used on the net section, as designed on the Aqueduct, has adequate tensile strength up to heads of 144 feet. As the strength of the concrete pipe is based wholly on the tensile strength of the reinforcing steel used in its construction, and not on the strength of the concrete in tension, there would have to be as much weight of steel in a concrete pipe for a 144-foot head, as in a steel pipe, and the cost of the concrete is extra cost for concrete pipe above that of steel pipe for this head.

A series of tests of concrete pipe were made for the Reclamation Service in Los Angeles by J. H. Quinton in 1915. The result of these experiments and tests is shown in Water Supply and Irrigation Paper No. 143, of the U. S. Geological Survey. This experimental pipe all had a diameter of 5 feet, a length of section of 20 feet, and a thickness of 6 inches. A number of different waterproofing compounds were used, and the pipes were all plastered. A mixture as rich as 1 of cement, 2 of sand, and 4 of gravel was used. Mr. Quinton says on page 56: "Do not use steel concrete pipes for heads over 70 feet, except for short distances, where a 100-foot head might be used by taking special precautions."

The forms used in the manufacture of this pipe were developed for this particular piece of work. It was necessary to have a collapsible form that could readily be dismounted and loaded on to a car, which was pushed along on the inside of the pipe. This form proved entirely satisfactory and was used throughout for the construction of all the pipes. The panels were sheeted with thin galvanized iron in order to protect the lumber, which would otherwise curl up and split.

The concrete ends of siphon No. 9, and all of Nos. 10, 11, 11-A, 12, and 13, were made of Tufa



CONSTRUCTION QUANTITIES

Concrete 1.24 Cu. Yds. per. Lin. Ft.

Longitudinal Reinforcement
17–3/4" diam. rods 36'-11" long, lapped 3'-2", spaced ab't. 2'-0"

Transverse Reinforcement Spaced 4" c. to c.

Head	Rods	Length
70'	3/4" diameter	36'-11"
45'	5/[illegible]" "	36'-5"
30'	1/2" "	35'-10"
15'	3/8" "	35'-4"

FIG. 32—TYPICAL DETAIL OF CONCRETE PIPE—LOS ANGELES AQUEDUCT.

cement from the Fairmont Mill. The other pipes were made of straight cement. It is believed that the Tufa cement pipe is more satisfactory than that made with straight cement, because the Tufa cement is ground finer, and therefore should be denser than the straight cement.

The mixtures used in the manufacture of this pipe were 1 of cement, 2 of sand, and 4 of stone. In pipes Nos. 20 and 21, river-washed gravel was used, which would leave about 50 per cent of stone on a quarter-inch screen. The thickness of the shell on the sides and top was 9 inches. With a head of 70 feet, this would give a tension of about 200 pounds per square inch on the concrete shell. From tests made with the materials used, it was found that the concrete, after attaining an age of one month, was fully strong enough, or stronger than required to resist this tension without any of the load coming on the steel.

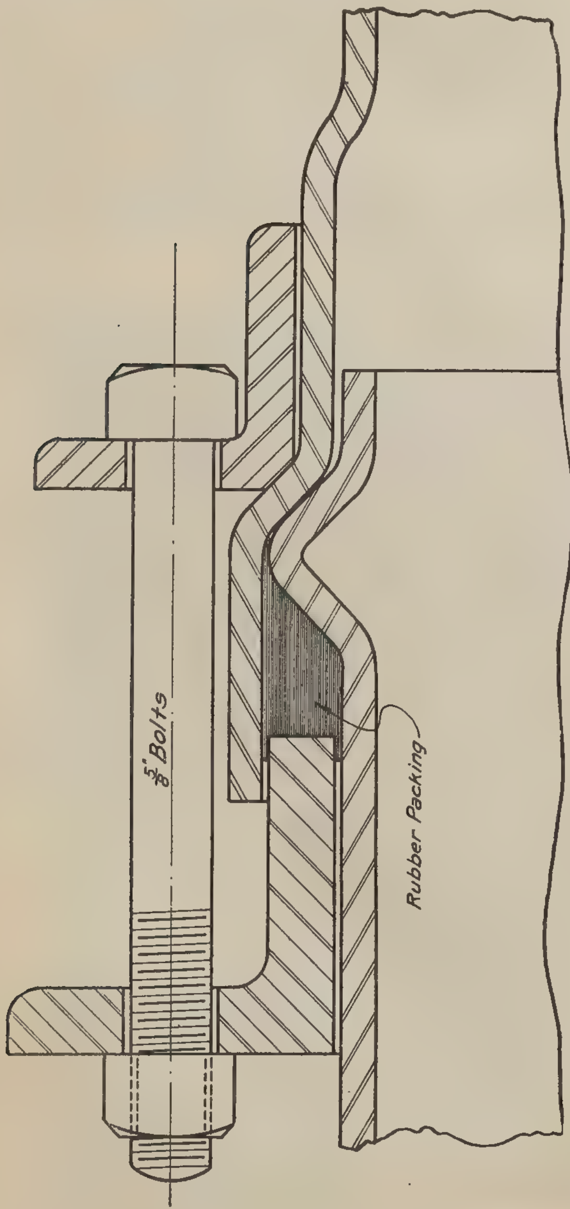


FIG. 33—HIGH PRESSURE BOLTED SOCKET SPIGOT END JOINT (PATENTED)—AMERICAN SPIRAL PIPE WORKS.

Reinforcement

The steel rods were placed 1/3 inch from the outside edge of the pipe. They were lapped 18 inches, and wired together. After the wet concrete was cast around these steel rods, the steel was put under compression by the shrinkage of the concrete in setting. When the load came on, it was changed from compression into tension, and it was expected that there would be enough movement in this process to produce slight horizontal ruptures in the concrete. No such ruptures, however, were found in any of the pipes, as had been the case with other large concrete pipes made with thin shells, where spouting jets of water occurred along longitudinal cracks. For these reasons, it is believed that the 9-inch shell of concrete is itself carrying the load, and that the steel has not been placed in tension. The concrete pipes have all been buried in trenches and covered with soil. Where possible these trenches were excavated with steam shovels, as in the Antelope Valley siphon. It is good practice so to bury the pipe in order to protect it against drying out and temperature movements.

Concrete pipes should, where possible, be built in cool weather so that it may be put under a condition of compression due to subsequent temperature changes, as concrete is about ten times as strong under compression as under tension. Care was exercised in keeping the pipes moist by wetting down the backfilled ground, and keeping it moist during the curing of the pipe. During this period, the ends of the pipe were kept closed with curtains, in order to maintain a humid atmosphere in the pipe. As soon as the pipe attained a sufficient age, as from a month to six weeks, it was slowly filled with water and kept full. In one pipe a circular crack opened at the bottom of the siphon. A leak of 6 to 8 gallons per minute developed before the pipe was completely filled with water. Instead of drawing off the water from the pipe, it was allowed to stand full for a week or ten days. The crack entirely closed with the swelling of the concrete, and no further trouble was experienced with it.

In the case of the Elsmere siphon, a leak of about five gallons a minute came from near the bottom of the pipe. The pipe was emptied and it was found that there was a defect in the concrete. This was cut out, and plugged with a small amount of new concrete and the leak was stopped at little expense.

Expansion Joints

The first concrete pipes filled with water were in the Whitney and Elsmere canyons. These pipes have heads of 20 and 75 feet respectively. Two expansion joints were put in each of these pipes. They were of the "Z" type and the joints were filled with asphalt. Slight leaks occurred at these joints, and no movements could be observed, due to ex-

pansion or contraction. It was thereafter concluded that the expansion joints were unnecessary, and none of the other concrete pipes had any expansion joints in them. Practically no trouble was experienced with any of the concrete pipes that were filled with water soon after their construction.

The Antelope Valley siphon crosses a valley with a total length of 22,746 feet. This valley has gently sloping sides. The north end of the siphon for a distance of 2,736 feet is a concrete pipe which has a maximum head of 75 feet. It is here connected with a steel pipe, which has a maximum head of 200 feet. At the south end there is 2,314 feet of concrete pipe, with a maximum head of 75 feet. Because of the unavoidable schedule of the work, this concrete pipe was built first, and the intervening steel pipe was not constructed for a year afterwards. It was not possible, therefore, to fill promptly the concrete pipe, and there was a scarcity of local water to adequately keep the ground over the concrete thoroughly soaked. Curtains were hung over the ends of the concrete pipe. The pipe, unfortunately, was built during the summer time instead of during cool weather. It was, therefore, subjected to a double shrinkage, due to temperature when cool weather did arrive and to drying out. This is an intensely arid region, with a relative humidity of about 20, and the summer temperatures exceed 100 degrees Fahrenheit. Six months after the concrete pipe was completed, a number of cracks opened in the concrete. These cracks ranged all the way from hair lines to openings as great as ⅛ of an inch. They occurred at intervals of 40 or 50 feet in some places, and in all, aggregated perhaps 50 cracks. The Superintendent who built the pipe was much disturbed over these cracks, and began cutting into the pipe dovetail openings about one inch deep, ½ inch wide at the inner surface of the pipe, and about ¾ inch wide at the base of the cut. These were then plugged with a cement mortar. After a few of them had been so plugged, the work was stopped, as it was considered undesirable to cut into the shell of the pipe, and because it was believed that the pipe would expand, and close the cracks when it was filled with water.

After the steel pipe was laid and connected with the concrete, the pipe was slowly filled, the local water supply being limited, and the leakage being as great as the inflow for several months. As the various sections of the Aqueduct became connected up, more water was available and the pipe was at last filled, after it was over a year old. Quite a large number of leaks of considerable size, sufficient to make small boiling springs along the pipe, at first were developed. They perceptibly became less as the pipe soaked up, and within a month after the pipe had been filled, it was entirely tight. A head of water was also accumulated at the upper end of the siphon by building a dirt dam in the pipe, and then suddenly the dam was broken, and the mud and silt sluiced through the pipe, and out the lower end. This may have facilitated the closing of the opening.

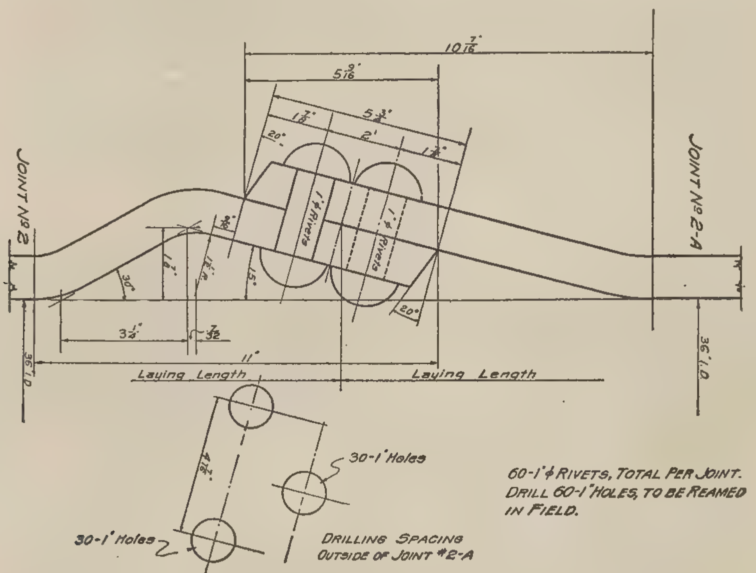


FIG. 34—TYPICAL DOUBLE RIVETED BUMP JOINT.

No longitudinal cracks ever opened in any of these pipes. Longitudinal cracks of this nature do not close with expansion of the concrete, as in the case of the circular cracks. On other works, where these longitudinal cracks occur, the method adopted for closing them is to keep the full head on the pipe, drive thin steel wedges into the crack so as to hold it apart, then take the water out of the pipe so that the crack can be grouted both inside and out, allow the cement to harden, and then refill the pipe.

Transition Joints

The casting of a concrete joint, connecting a steel pipe with a concrete pipe under 75 feet of pressure, is not entirely a satisfactory practice. In the case

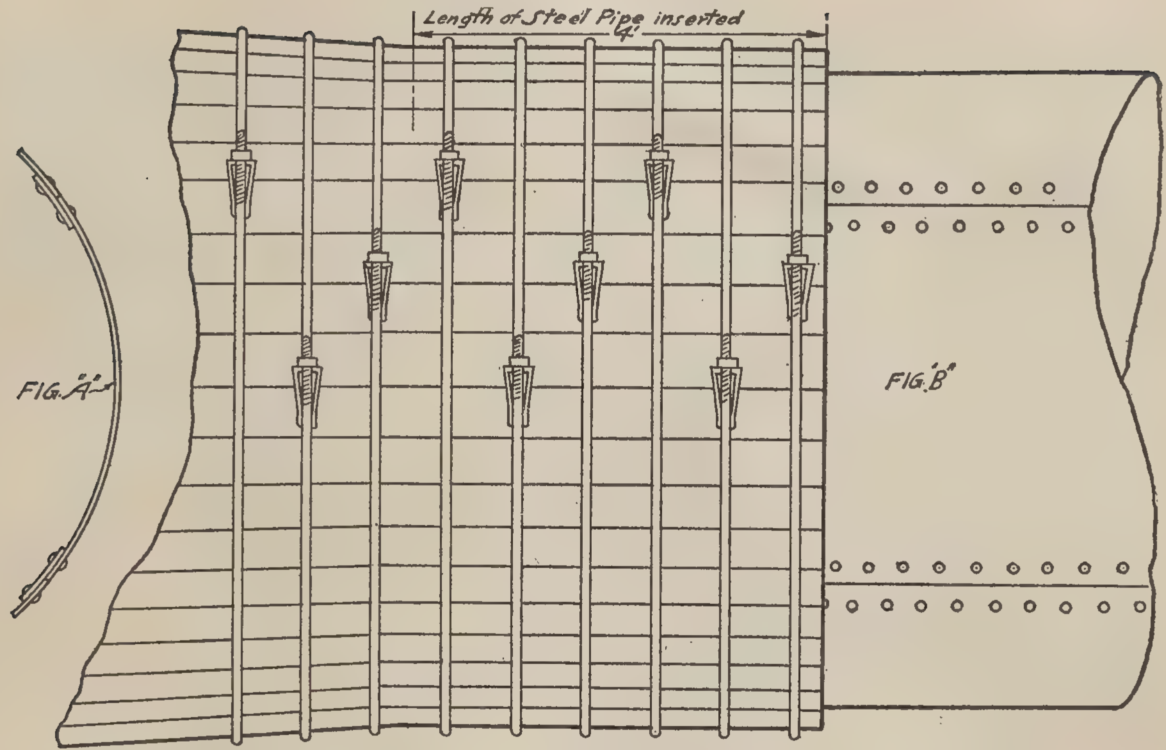


FIG. 35—DETAIL OF CONNECTION OF 8-FT. WOOD STAVE TO 6-FT. STEEL PIPE—PACIFIC GAS & ELECTRIC COMPANY.

Fig. "A"—Detail of Steel Pipe Showing Joints. Butt Joints with inside butt plates. Outside heads of rivets flattened over end of Steel Pipe to be inserted into Wood Stave Pipe.

Fig. "B" shows the 6-ft. end of a taper section of Wood Stave Pipe, reducing from 8 ft. to 6 ft. diameter, and illustrates the method of connecting to Steel Pipe.

PACIFIC TANK & PIPE COMPANY
SAN FRANCISCO
LOS ANGELES PORTLAND
CONNECTION
8-ft. Wood Stave to 6-ft. Steel.
Date 6-3-13
Scale ¾ in.-1 in.

of the Deadman siphon, a 10-foot concrete pipe is connected with an 11-foot steel pipe at a point where the pressure head is 70 feet. An expansion joint was made at this connection, the concrete overlapping the steel for 6 feet, or the width of one plate, and the annular opening of ¾ inch between the steel and concrete caulked with oakum. It was difficult to make this opening tight. Threaded steel rods were cast in the concrete, so as to project about 6 inches; a circular piece of angle iron was then fitted over the steel, and holes bored in it to fit the threaded rods. The annular opening between the steel and the concrete was then caulked with oakum, and the steel angle drawn down against the oakum with nuts. It was observed that this joint was tight, as the pipe filled up, until there was about 30 feet of head on the joint. It then gradually leaked more and more until a head of 70 feet was on the pipe, when the leakage amounted to about 10 gallons per minute. Apparently the steel pipe changed its form with the increased head, and so caused the leakage. By a repeated re-caulking under pressure, the leak was finally eliminated.

Similar joints were at first cast between the steel and the concrete on the Antelope Valley siphon, but after the experience on the Deadman siphon, it was decided to make a rigid connection between the concrete and steel, and this was done by first riveting angle irons to the steel pipe and then casting a large block of concrete so as to envelope the steel pipe. This rigid joint proved satisfactory.

Leakage

In the case of the Whitney siphon, when the pipe was filled, the leakage was at the rate of about 4,000 gallons per day. The expansion joints were then cut out and plugged with cement. Three months after the leakage had dropped 320 gallons per day. This measurement was made volumetrically, as no water was admitted to the pipe during the test. The heads ranged from 60 to 70 feet.

An interesting comparison is made with the San Antonio steel siphon, which is constructed of ¼-in. steel plate, and was laid in hot weather. The pipe expanded and contracted considerably during its construction. The pipe was caulked, and no seepage was shown on the surface of the ground. The length of this pipe is 695 feet, or 73 per cent of the length of the Whitney concrete pipe. Its diameter is 9 feet, and the maximum head 72 feet. On September 27, 1910, this pipe was leaking at the rate of 4,577 gallons per 24 hours; on October 1, 3,235

gallons; on October 9, 2,400 gallons, and on November 27, it was practically tight.

Riveted Pipe

On account of the reduction in the cost of welded pipe there is a general tendency to use it in preference to riveted pipe. A great quantity of riveted pipe is still used, however, for the larger and lighter sections of the penstock.

Tables of Riveted Joints and General Specifications are included which are believed to be representative of the best practice.

SPECIFICATIONS FOR THE FABRICATION OF RIVETED STEEL PIPE

(A) GENERAL SPECIFICATIONS:

1. Plates:

(a) All plates shall be free from laminations or surface defects and be up to gage on the edges, standard variations being allowed.

(b) Any plate that develops defects during the process of punching, bending, and riveting incident to fabrication and erection of the pipe shall be rejected notwithstanding that the same may previously have satisfactorily passed specified test.

2. Length of Sections:

In general riveted pipe shall be made in three course sections, the total length of each section to be from 20 to 24 feet.

3. Joints:

Details of all joints shall be in accordance with the standards shown on the accompanying drawings. All roundabout lap joints shall be constructed with female end uphill.

Longitudinal lap joints shall point down and shall be located alternately 30 deg. to the left and to the right of the top center line of the pipe.

Longitudinal butt joints shall be located at the top center of the pipe except where angle sections, air valves, or manholes occur, in which cases they shall be located near the horizontal diameter.

Butt straps for the roundabout joints shall be riveted to the pipe in the shop for one half of the circumference less two rivets. The remaining portion is to be sub-punched but not reamed, and securely bolted for shipment.

All joints shall form a tight fit with each other. All angular joints shall be shop closed.

4. Angle Sections:

(a) Where angles or curves occur in either the alignment or the grade of the conduit, the plates must be cut and punched to the required lines for forming a small oblique angle at the roundabout seams, embracing as many courses as may be required to procure the total deflection or curvature, the courses being put together with the longitudinal seams staggered.

(b) In general the deflection angle formed by two consecutive courses may range from one (1) to five (5) degrees in the plane of the bend, according to locality, but greater deflection angles shall not be made except as specifically authorized by the Company. In general, no angle section shall consist of less than three courses.

(c) The work must be so laid out as to bring all longitudinal seams as near the horizontal diameter of the pipe as possible, preferably in the upper half of the section.

5. Taper Courses:

In forming taper courses, the plates must be cut and punched to the required lines along the four edges, so as to bring the pitch lines of the rivets in the roundabout seams into planes parallel with each other and at right angles to the axis of the section.

6. Rivet Pass Holes:

A rivet pass hole shall be located in each section on the top center line of the pipe about 3 ft. uphill from the field roundabout joint. Pass holes shall be tapped and provided with brass plugs.

7. Marking:

(a) The sections of the penstock, together with all special material shall be carefully marked for identification in the field, in accordance with an erection diagram to be furnished by the Contractor, for field use.

(b) Each shop assembled length shall be plainly marked at its lower end inside and on top outside with its number clearly painted. Two clear and distinguishable center punch witness marks shall be placed on the top outside of each length to identify corresponding rivet holes. The same rivet holes shall be further distinguishable by two clean paint marks.

(B) WORKMANSHIP:

8. General:

All workmanship shall be first class and in accordance with the best American shop practice. All pieces of the same mark shall be interchangeable. All sections of pipe, except taper pieces, shall be true circles of the required internal diameters.

9. Shearing:

Shearing shall be neatly and accurately done, and all portions of the work exposed to view shall be neatly finished. The cuts shall be clean, without drawn or ragged edges and without splitting away from the sheared edge.

10. Planing:

Edges of plates forming longitudinal seams for butt strap pipe shall be planed to bring the pipe to exact diameter and to insure tightness of the joints. The ends of all sections shall be properly cut to true lines.

11. Beveling and Scarfing:

(A) Lap Joint Pipe:

In lap work, the edges of all plates must be properly cut or sheared to true lines and all edges which are to be caulked in the finished pipe shall be properly beveled on a plane at approximately 70 deg. with the plane of the plate. At the end of each course where the lap of the longitudinal seam occurs the plate must be reduced by planing or hammering, or both, to a fine edge to which one of the rivets of the round seam must be driven to insure tightness.

(B) Butt Joint Pipe:

(a) The edges of all butt straps shall be properly beveled for outside caulking in the field.

(b) Roundabout seams of the shell plates shall not be beveled for inside caulking, but the plates shall be cut square and so designed as to come within ¼ in. of each other.

(c) Wherever joints require scarfing, such shall be properly done to a fine edge and rivets properly spaced at such points to insure water tightness.

STANDARD DETAILS FOR RIVETED JOINTS FOR STEEL PIPE
ASSUMPTIONS USED IN DESIGN
PACIFIC COAST ELECTRICAL ASSOCIATION

MATERIALS

Plate shall conform to Standard Specifications of American Society for Testing Materials for boiler flange steel.

Rivets shall conform to Standard Specifications of American Society for Testing Materials for boiler rivet steel.

STRESSES

Type	*Ultimate Strength*
Tension	55,000 lbs. per sq. in.
Shear	44,000 lbs. per sq. in.
Bearing	95,000 lbs. per sq. in.

DETAILS

Hole diameter shall be 1/16 inch larger than shank of cold rivet, and is used in computing shearing and bearing strengths of rivets.

Punching and Reaming. All holes for butt-joint pipe shall be sub-punched and reamed. All holes for lap-jointed pipe of 7/16 inch thickness or more shall be sub-punched and reamed. Holes in lap-joint pipe of ⅜ inch thickness or less may be punched to size.

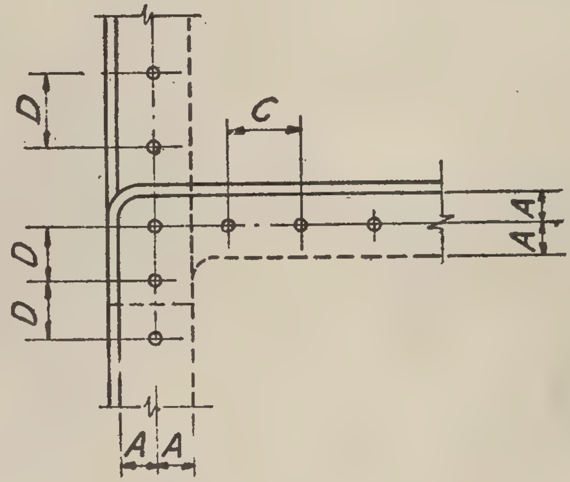
Deductions for Net Area. For punched holes, a deduction for hole of 3/16 inch greater diameter than cold rivet shank diameter is made in computing net area. For sub-punched and reamed holes, a deduction for hole of 1/16 inch greater diameter than cold rivet shank diameter is made in computing net area.

Edge Distances. Edge distances are at least 1.5 times diameter of hole.

Rivet Spacing. The distances between rows of rivets is such that sum of the two net diagonal dimensions between holes will not be less than 1.25 times the net distance between holes on gauge lines.

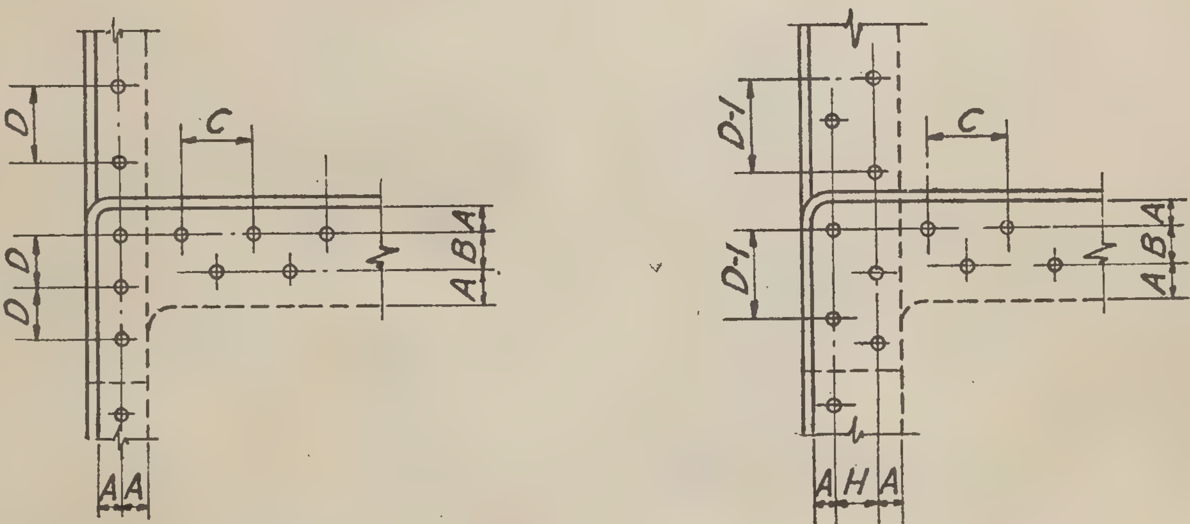
The maximum spacing of holes along caulked edges is governed by the formula $P = 2\frac{1}{2}\, t + d + 1\frac{1}{2}$ inch, where t=plate thickness, d=dia. of rivet hole, p=pitch.

All rivet spacings shall be great enough to permit of use of standard rivet dies.



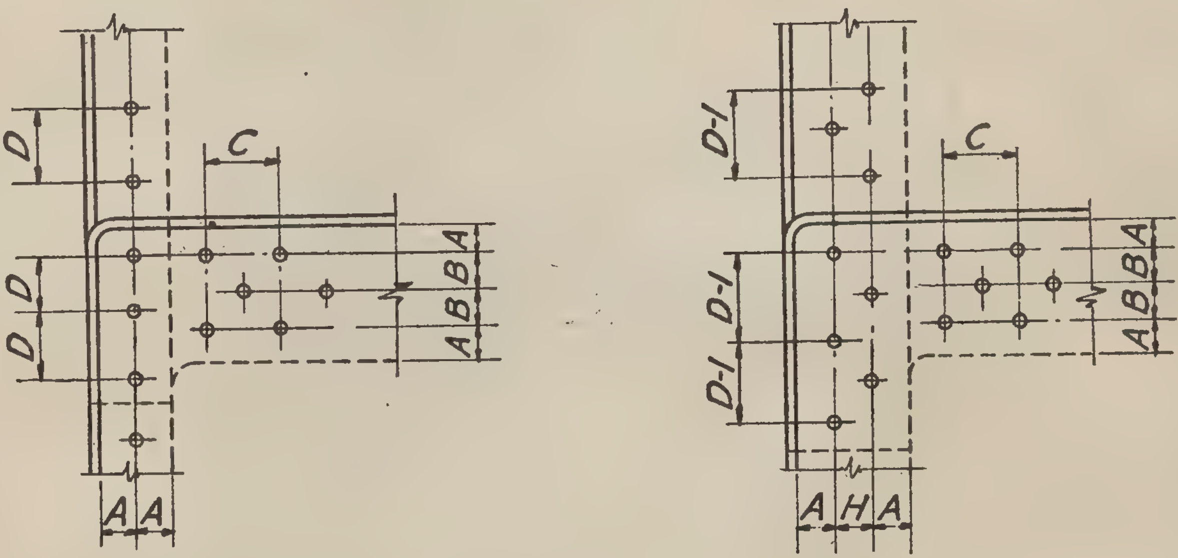
STANDARD DETAILS FOR RIVETED JOINTS FOR STEEL PIPE
SINGLE RIVETED LAP JOINTS
PACIFIC COAST ELECTRICAL ASSOCIATION

Plate Thick.	Dia. Rivet	A	C	D	Eff. Plate	Eff. Riv. B.	Eff. Riv. S.
1/8	3/8	11/16	1 5/16	1 5/16	57.3	57.8	73.5
3/16	1/2	7/8	1 3/4	1 3/4	60.6	55.6	60.5
1/4	5/8	1 1/16	2	2	59.3	59.3	59.3
5/16	3/4	1 1/4	2 1/4	2 1/4	58.3	62.2	58.8
3/8	7/8	1 7/16	2 5/8	2 5/8	60.0	61.1	55.5
7/16	7/8	1 7/16	2 5/8	2 5/8	64.3	52.8	48.0
1/2	1	1 5/8	3	3	64.5	61.1	47.2
9/16	1 1/8	1 13/16	3 3/8	3 3/8	64.8	60.8	46.7
5/8	1 1/4	2	3 3/4	3 3/4	64.9	60.5	46.2



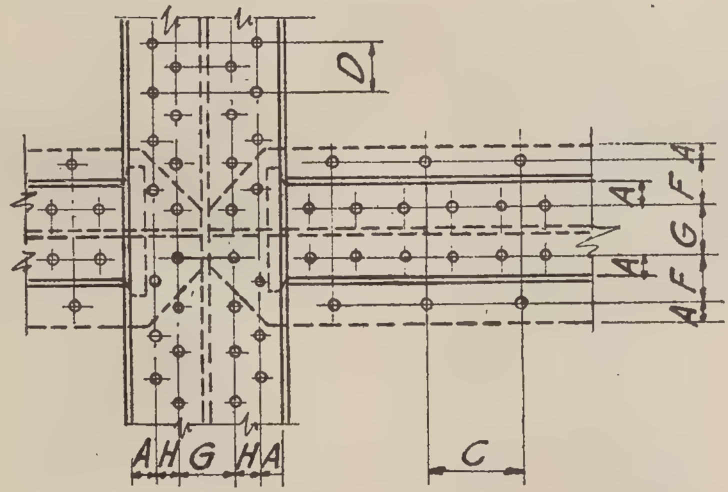
STANDARD DETAILS FOR RIVETED JOINTS FOR STEEL PIPE
DOUBLE RIVETED LAP JOINTS
PACIFIC COAST ELECTRICAL ASSOCIATION

Plate Thick.	Dia. Rivet	A	B	C	D	D-1	H	Eff. Plate	Eff. Riv.B.	Eff. Riv.S.
1/8	3/8	11/16	1 1/16	2 1/16	1 5/16			72.8	73.3	93.2
3/16	1/2	7/8	1 1/4	2 1/2	1 3/4			72.6	77.8	84.6
1/4	5/8	1 1/16	1 7/16	2 13/16	2			71.3	84.5	84.5
5/16	3/4	1 1/4	1 11/16	3 1/16	2 1/4			69.5	91.7	86.7
3/8	3/4	1 1/4	1 3/4	3 1/8	2 1/4			70.0	90.0	70.7
7/16	7/8	1 7/16	2	3 7/16		3 7/16	2	72.5	94.0	73.4
1/2	1	1 5/8	2 3/8	3 13/16		3 13/16	2 3/8	72.2	96.1	74.5
9/16	1 1/8	1 13/16	2 11/16	4 1/16		4 1/16	2 11/16	70.7	101.0	77.4
5/8	1 1/8	1 13/16	2 11/16	4		4 3/16	2 11/16	70.3	106.6	70.7
11/16	1 1/4	2	3	4 1/2		4 1/2	3	70.8	100.6	69.7
3/4	1 1/4	2	3 1/8	4 3/16		4 5/8	3	68.7	108.0	68.8



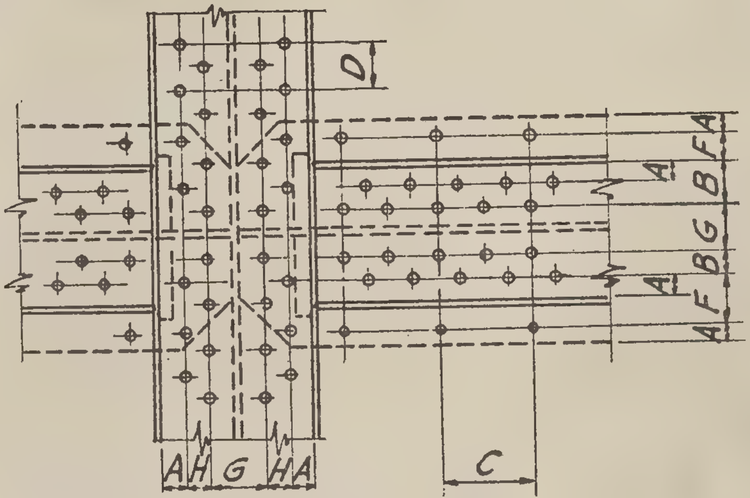
STANDARD DETAILS FOR RIVETED JOINTS FOR STEEL PIPE
TRIPLE RIVETED LAP JOINTS
PACIFIC COAST ELECTRICAL ASSOCIATION

Plate Thick.	Dia. Rivet	A	B	C	D	D-1	H	Eff. Plate	Eff. Riv.B.	Eff. Riv.S.
1/4	1/2	7/8	1 5/16	2 11/16	2			74.5	108.0	88.7
5/16	5/8	1 1/16	1 5/8	3	2 1/4			73.0	118.6	95.0
3/8	5/8	1 1/16	1 1/2	3 1/16	2 5/8			73.5	116.4	77.5
7/16	3/4	1 1/4	1 3/4	3 7/16		3 7/16	2	76.2	125.0	82.8
1/2	3/4	1 1/4	1 3/4	3 5/16		3 1/2	2	76.0	127.0	75.2
9/16	7/8	1 7/16	2	3 13/16		3 13/16	2	75.5	127.4	77.2
5/8	7/8	1 7/16	1 15/16	3 9/16		4	2 1/16	73.7	133.0	74.3
11/16	1	1 5/8	2 3/16	4 1/8		4 1/4	2 1/4	74.2	133.0	75.0
3/4	1 1/8	1 13/16	2 1/2	4 9/16		4 9/16	2 1/2	73.9	134.7	77.5
13/16	1 1/8	1 13/16	2 1/2	4 1/2		4 11/16	2 7/16	73.3	136.5	72.4
7/8	1 1/4	2	2 13/16	5		5	2 13/16	73.6	136.0	74.2
15/16	1 1/4	2	2 7/8	4 3/4		5 1/8	2 3/4	72.4	143.0	73.0
1	1 1/4	2	3	4 9/16		5 1/4	2 3/4	71.3	149.0	71.3



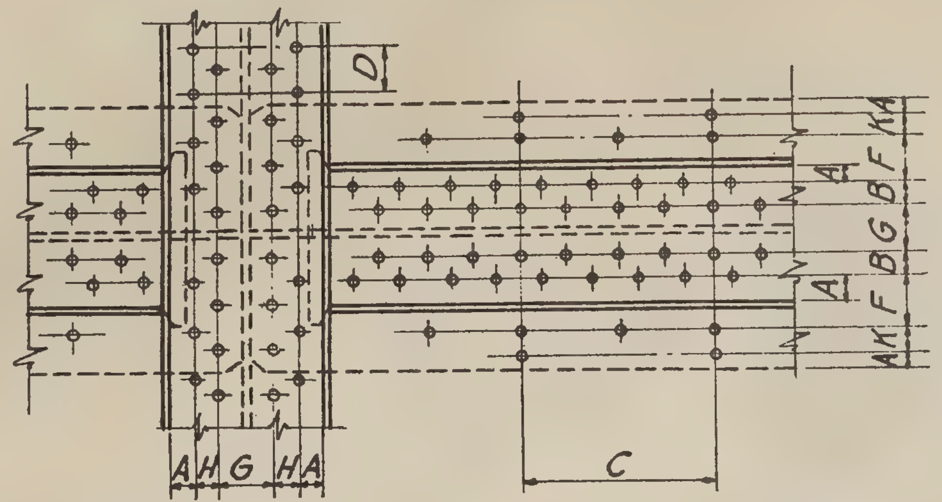
STANDARD DETAILS FOR RIVETED JOINTS FOR STEEL PIPE
DOUBLE RIVETED DOUBLE BUTT JOINTS
PACIFIC COAST ELECTRICAL ASSOCIATION

Thickness											Efficiency		
Pipe Plate	Long. Strap	Cir. Strap	Dia. Riv.	A	B	C	D	F	G	H	Out. Row	Inside Row	Rivet
3/8	5/16	5/16	3/4	1 1/4	1 15/16	4 3/4	3 1/8	2 11/16	2 3/4	1 5/8	83.0	89.0	82.4
7/16	3/8	3/8	7/8	1 7/16	2 1/16	5 7/16	3 3/8	3	3 1/8	1 7/8	82.8	86.9	82.8
1/2	7/16	7/16	1	1 9/16	2 9/16	6 1/8	3 5/8	3 5/16	3 3/8	2 7/16	82.6	88.5	83.1
9/16	1/2	1/2	1 1/8	1 3/4	2 15/16	6 7/8	3 15/16	3 11/16	3 3/4	2 3/4	82.8	88.4	82.5
5/8	1/2	1/2	1 1/4	2	3 1/4	7 9/16	4 1/16	4 1/16	4 1/4	3 3/16	82.6	88.0	82.7
11/16	9/16	9/16	1 1/4	2	3 1/4	7 7/16	4 1/4	4 1/16	4 1/4	3 1/8	82.2	85.8	82.1
3/4	9/16	9/16	1 1/4	2	3 5/16	7 1/4	4 1/4	4 1/16	4 1/4	3 1/8	81.9	83.5	82.5
13/16	5/8	5/8	1 1/4	2	3 5/16	7 3/16	4 3/8	4 1/8	4 1/4	3 1/16	81.6	82.0	81.6
7/8	5/8	5/8	1 1/4	2	3 5/16	7 1/8	4 3/8	4 1/8	4 1/4	3 1/16	81.6	80.5	80.8
15/16	3/4	3/4	1 1/4	2	3 5/16	7 1/8	4 5/8	4 3/16	4 1/4	3	81.5	79.0	79.7
1	3/4	3/4	1 1/4	2	3 5/16	7	4 5/8	4 3/16	4 1/4	3	81.1	78.0	77.4



STANDARD DETAILS FOR RIVETED JOINTS FOR STEEL PIPE
TRIPLE RIVETED DOUBLE BUTT JOINTS
PACIFIC COAST ELECTRICAL ASSOCIATION

Thickness											Efficiency		
Pipe Plate	Long. Strap	Cir. Strap	Dia. Riv.	A	B	C	D	F	G	H	Out. Row	Inside Row	Rivet
3/8	5/16	5/16	5/8	1	1 1/2	6	3	2 3/8	2 1/4	1 3/8	88.5	90.4	92.5
7/16	3/8	3/8	5/8	1	1 1/2	6 1/8	2 3/4	2 3/8	2 1/4	1 1/2	88.8	88.6	88.5
1/2	7/16	7/16	3/4	1 1/4	1 3/4	6 7/8	3 1/8	2 3/4	2 3/4	1 5/8	88.4	88.5	93.3
9/16	1/2	1/2	7/8	1 7/16	1 7/8	7 3/8	3 1/4	3 1/8	3 1/8	2 1/16	87.2	87.8	100.0
5/8	1/2	1/2	7/8	1 7/16	1 7/8	7 3/8	3 1/4	3 1/8	3 1/8	2 1/16	87.4	86.4	99.4
11/16	9/16	9/16	1	1 9/16	2 1/4	8	3 5/8	3 3/8	3 3/8	2 7/16	86.9	86.5	100.0
3/4	9/16	9/16	1 1/8	1 3/4	2 5/8	8 1/4	4 1/8	3 3/4	3 3/4	2 11/16	85.6	85.5	100.0
13/16	5/8	5/8	1 1/4	2	3 1/16	8 3/4	4 3/8	4 1/8	4 1/4	3 1/16	85.0	85.4	100.0
7/8	5/8	5/8	1 1/4	2	3 1/16	8 3/4	4 3/8	4 1/8	4 1/4	3 1/16	85.0	84.0	100.0
15/16	3/4	3/4	1 1/4	2	2 15/16	9 3/8	4 1/4	4 3/16	4 1/4	3 1/8	86.0	84.5	100.0
1	3/4	3/4	1 1/4	2	2 15/16	9 3/8	4 1/4	4 3/16	4 1/4	3 1/8	86.3	83.6	100.0
1 1/16	7/8	7/8	1 1/4	2	2 13/16	10	3 3/4	4 1/4	4 1/4	3 1/4	86.9	84.0	91.8
1 1/8	7/8	7/8	1 1/4	2	2 13/16	10	3 3/4	4 1/4	4 1/4	3 1/4	86.7	83.5	86.6
1 3/16	1	1	1 1/4	2	2 13/16	10	3 3/4	4 3/8	4 1/4	3 1/4	86.7	82.8	82.2
1 1/4	1	1	1 1/4	2	2 7/8	9 1/2	3 3/4	4 3/8	4 1/4	3 1/4	86.4	81.6	82.1



STANDARD DETAILS
FOR RIVETED JOINTS FOR STEEL PIPE QUADRUPLE RIVETED
DOUBLE BUTT JOINTS
PACIFIC COAST ELECTRICAL ASSOCIATION

Thickness												Efficiency			
Pipe	Long. Strap	Cir. Strap	Dia. Riv.	A	B	C	D	F	G	H	K	1st Row	2nd Row	3rd Row	Rivet
3/8	5/16	5/16	5/8	1	1 1/2	12	3	2 3/8	2 1/4	1 3/8	2	94.0	93.6	94.4	100.0
7/16	3/8	3/8	3/4	1 1/4	1 11/16	13	3 1/8	2 3/4	2 3/4	1 5/8	2 1/4	93.6	94.6	96.7	100.0
1/2	7/16	7/16	3/4	1 1/4	1 3/4	13 1/2	3 1/8	2 3/4	2 3/4	1 5/8	2 1/4	93.8	94.0	94.2	100.0
9/16	1/2	1/2	7/8	1 7/16	1 7/8	14 3/4	3 1/4	3 1/8	3 1/8	2 1/16	2 5/8	93.6	93.8	94.5	100.0
5/8	1/2	1/2	7/8	1 7/16	1 7/8	14 3/4	3 1/4	3 1/8	3 1/8	2 1/16	2 5/8	93.6	93.0	92.5	100.0
11/16	9/16	9/16	1	1 9/16	2 1/4	16	3 5/8	3 3/8	3 3/8	2 7/16	3	93.4	93.1	92.7	100.0
3/4	9/16	9/16	1 1/8	1 3/4	2 3/4	16 1/4	4 1/8	3 3/4	3 3/4	2 11/16	3 3/8	92.6	92.5	92.5	100.0
13/16	5/8	5/8	1 1/8	1 3/4	2 5/8	17	4 1/4	3 13/16	3 3/4	2 5/8	3 3/8	93.2	92.5	91.3	100.0
7/8	5/8	5/8	1 1/4	2	3 1/16	17 1/2	4 3/8	4 1/8	4 1/4	3 1/16	3 3/4	92.4	92.0	91.2	100.0
15/16	3/4	3/4	1 1/4	2	2 15/16	18 3/4	4 1/4	4 3/16	4 1/4	3 1/8	3 3/4	93.1	92.2	90.4	100.0
1	3/4	3/4	1 1/4	2	2 15/16	18 3/4	4 1/4	4 3/16	4 1/4	3 1/8	3 3/4	93.1	91.7	89.4	100.0
1 1/16	7/8	7/8	1 1/4	2	2 13/16	20	3 3/4	4 1/4	4 1/4	3 1/4	3 3/4	92.6	91.9	89.0	97.0
1 1/8	7/8	7/8	1 1/4	2	2 13/16	20	3 3/4	4 1/4	4 1/4	3 1/4	3 3/4	93.1	91.5	88.0	91.3
1 3/16	1	1	1 1/4	2	2 13/16	20	3 3/4	4 3/8	4 1/4	3 1/4	3 3/4	93.3	91.3	87.4	86.6
1 1/4	1	1	1 1/4	2	2 7/8	19 1/4	3 3/4	4 3/8	4 1/4	3 1/4	3 3/4	93.0	91.9	86.0	85.4

12. Punching and Reaming:

(a) Punched holes shall be accurately spaced, true to line, so that when plates are brought together, holes shall exactly match.
(b) Only the sharpest dies and punches shall be used. The diameter of the die must never exceed the diameter of the punch by more than 3/32 of an inch.
(c) The use of drift pins will be permitted only for drawing the material together. No drifting to enlarge unfair holes will be allowed. Necessary corrections shall be made with a reamer. Poor matching of punched holes will be sufficient cause for rejection.
(d) All rivet holes for butt joint pipe and for lap joint pipe where the plate thickness exceeds 3/8 in. shall be sub-punched 1/8 in. and reamed to size. The diameter of the finished rivet hole shall be 1/16 in. greater than the diameter of the rivet as shown on the drawings.
(e) Rivet holes for lap joint pipe with a plate thickness 3/8 in. or less shall be punched to finished diameter without reaming.

13. Rolling:

All plates shall be bent cold, to a true circle of the specified diameter of the pipe, as nearly as practicable, by the use of a template.

14. Drifting:

No drifting to rectify unfair holes will be allowed. If holes require enlargement to admit the rivet or bolt, it must be reamed, and under no circumstances is the metal in the vicinity of the hole to be distorted or injured. The use of drift pins will be allowed only for bringing together the several parts forming a member and they shall not be driven with such force as to injure the adjacent metal.

15. *Riveting:*

(a) The size of rivets called for on the drawings shall be understood to mean the actual size of the rivets before heating.
(b) Before riveting, all plates must be thoroughly cleaned and freed from rust and scale. Burrs shall be removed.
(c) Wherever possible, rivets shall be driven by pressure tools of sufficient capacity to upset the metal, exerting a slow and steady pressure of not less than fifty (50) tons for rivets of 3/4-in. diameter nor less than seventy (70) tons for larger diameter, and retaining this pressure while the rivet head is being formed.
(d) All rivets, after driving, shall completely fill the hole, and have full heads concentric with the shank. No recupping nor caulking of heads will be allowed. All loose, burned or otherwise defective rivets shall be cut out and replaced, great care being exercised not to injure the adjacent material, drilling out if necessary.
(e) In order to avoid shrinkage of the rivets on cooling, it will be required that the riveting pressures be held for the following periods of time on each rivet:

1¼ in. diameter rivets	55	seconds
1⅛ in. " "	45	"
1 in. " "	35	"
⅞ in. " "	25	"
¾ in. " "	25	"
⅝ in. " "	20	"
½ in. " "	18	"

All rivets shall be cone head rivets.

16. Caulking:

All seams must be caulked on the outside in first-class boiler work fashion, and the inspection thereof completed before any coating is applied to the pipe.

Welded Pipe

The use of welded pipe for important penstocks is rapidly increasing. This is largely due to the great improvement in manufacturing methods insuring a very uniform product. For penstocks welded pipe is practically limited to that made by a forge welding process; autogenous welding is not considered reliable enough for this class of work.

In recent years the price of welded pipe has been greatly reduced, so that a pipe designed on an economic basis will consist of a great percentage of welded pipe. In fact, some lines that are now being built are of welded pipe only as an economic investigation showed that the welded pipe was cheaper to install than riveted. This can be accounted for easily by considering the greater carrying capacity of the smoother welded pipe and the reduced weight on account of the absence of butt straps and greater efficiency of the longitudinal joints.

Failures

Only one failure of recently made welded pipe has been recorded. An investigation showed that this failure was due to a flaw in a steel manhole casting that had been welded into the pipe. In this case no evidence whatever was found of any failure of the weld itself. The welding of steel castings should never be permitted, all fittings to be welded to the pipe should be of forged steel. Several failures of old pipe lines have been noted but these have been found to be due to the method of manufacture, which was not at all developed to the point that it is today.

Efficiency of Weld

It is universal practice to consider the efficiency of the weld as 90 per cent. This is the value guaranteed by the manufacturers; but innumerable tests show that the weld is practically as strong as the plate. Tests also show that the elastic limit of the steel is altered very little by welding so that it is proper to assume the elastic limit of the welded joint as 90 per cent of that of the original plate where the factor of safety is based on the elastic limit.

Joints

For all pipes 26 in. in diameter and above, the common type of "Bump Joint" is preferred. This joint is of especial value in laying the pipe as it permits of slight deviations in alignment in order to maintain the proper line. Bump joints can be either double or single riveted but double riveting is preferred. Rivet holes should be drilled 1/16 in. small in the shop and reamed to size in the field after the pipe is in place.

As it is not possible to rivet bump joints in pipes smaller than 26 in., below this diameter flange joints must be used.

WOOD STAVE PIPES IN OPERATION:

Plant	Year Installed	Diameter of Pipe	Thickness of Staves	Length of Pipe	Max. Static Head	Kind of Wood	Remarks
Cow Creek....	1907	42 in.	2¾ in.	781 ft.	40 ft.	Local red fir	B. F. C. C.
Electra	1900	48 "		2763 "	75 "	Redwood	B. F. C. C.
Halsey	1916	96 "	2½ "	1611 "	58 "	Redwood	P. E. C. on saddles
Inskip	1910	72 "	2¾ "	2145 "	97 "	Local red fir	B. F. C. C.
Stanislaus	1907	66 "	1⅞ "	1560 "	101 "	Local sugar pine	P. E. E. on saddles
Volta*	1901	36 "	2¾ "	3106 "	90 "	Local red fir	B. F. C. C.
Wise	1916	96 "	2½ "	1355 "	71 "	Redwood	P. E. C. on saddles

*This line replaced in 1921
P. E. C.—Pipe exposed completely
B. F. C. C.—Backfilled—completely covered
The red fir and sugar pine was heartwood, no sapwood being permitted.

Rivet Pass Holes

Some prefer to leave the rivet pass holes for the girth joints to be drilled in the field. In most cases, however, these are provided in the shop although there is no common practice regarding their location. In some cases they are placed in every length of pipe, three or four feet above the joint, and in others they are placed in every other length, a short distance below the joint. It is general practice to close the pass holes with malleable plugs.

Wood Stave Pipe

The general belief is that wood stave pipe should not be used in a permanent installation for greater heads than 100 feet. Instances have been given where small pipe has been used successfully on heads of 400 feet and 600 feet, but in general these are only for construction plants.

Wood stave pipe should be exposed and supported on concrete saddles, although in some cases it has been thought necessary to protect the pipe from the sun by building rubble walls on either side and filling in on top of the pipe with sand.

The following statement furnished by the Pacific Gas & Electric Company is representative of the best modern practice.

Data on Life of Wood Stave Pipe

The Volta penstock lines are the only ones that have been replaced, one being replaced in 1918 and the second line being replaced in 1921. These lines were leaking badly at that time.

The electra line has collapsed due to breaks in the welded pipe below, but the same material was used in the rebuilt pipe. The interior of the staves appears as sound as when new except where sapwood occurred. The bands have not yet been seriously weakened.

Disadvantage of Wood Stave Pipe

The experience on the larger wood stave pipe installation shows that if the pipe is emptied and then filled there is enough change in shape from the circular to cause some leakage through the staves.

Data on Latest Wood Stave Pipe Practice of P. G. & E. Co.

Materials

a. Staves—No. 1 Redwood—2½ in. thick dressed.
b. Bands—mild steel, stressed to 12,000 lb. per square inch maximum. Furnished in two pieces.
c. Tongue—⅛ in. x 1½ in. steel tongues joining staves.
d. Shoes—malleable iron of the Allen design.
e. Dipping and Painting—Shoes and bands are dipped in carbolineum before shipping and retouched with asphaltum paint where abraded.

Saddles

Our latest practice is to support the pipe on concrete saddles spaced about 10 ft. centers. These saddles support the pipe on a minimum arc length of 110 deg.

Protection Against Collapse

Air valves are placed at all convex grade changes to protect the pipe line from collapse.

Maintenance

The Halsey and Wise penstocks were sprayed with a cold preservative paint at the end of five years of service. This was done primarily to protect the bands, but also proved useful in penetrating the checks and cracks in the wood and practically stopping leakage.

Painting

The question of painting is one about which there is a great diversity of opinion, so it is not possible to give anything other than a few examples of what has been done.

Shop Coat

In many cases Detroit No. 500 graphite paint has been used, in others only pure hot boiled linseed oil. On a recent line Detroit Clear Liquid Primer No. 26 was used and found very satisfactory. The advantage of the linseed oil and the Clear Primer is that they are transparent, thus showing up all defects in the pipe and at the same time protecting against rusting during shipment.

Field Coat

Field painting has included Detroit Graphite, Dutch Boy Red Lead, Gas House Tar and hot asphaltum. There is no decided opinion regarding color, although in some cases there is a preference for a light gray on account of its better appearance and protection against excessive temperature when the pipe is empty. A light colored paint cannot be used over an asphaltum base.

The general practice is to paint the pipe both inside and outside, although there are instances where it was not considered necessary to paint the inside. There is also a decided feeling on the part of some companies that a thick coating such as hot dipping in asphaltum is not desirable on account of the possibility of deterioration of the steel under the coating without any visible effects on the surface.

The statement of the National Tube Company gives some interesting data on their painting practice.

Tests and Inspection

Shop Tests—Riveted Pipe

It is not considered practicable to subject riveted pipe to pressure tests in the shop. This is for the reasons that the strength cannot be quite accurately computed and the difficulty of conducting such tests on account of lack of equipment. With proper workmanship, which is, of course, subjected to rigid inspection, the variation in strength should be within well-defined limits. Material tests, of course, are made in accordance with standard practice.

Shop Tests—Welded Pipe

Every section of welded pipe should be tested to 150 per cent of normal working pressure. It is desirable to have this pressure applied suddenly several times, if possible, to attempt to produce the effect of water hammer. The weld should be hammered with a ten-pound sledge while under pressure. It is generally believed that the test pressure should be maintained for at least 15 minutes so as to be sure that no leaks develop. The test pressure to which each section is subjected should be stamped with a steel stamp on the pipe while it is in the testing machine.

If it can possibly be arranged, all bends should be tested the same as the straight pipe. Many times this is difficult to do and only an oil test can be made. This is conducted as follows: The pipe is heated and crude oil poured inside the section. If there is a defect in the pipe, the oil will either seep through to the outside or smoke will come through. This test is apparently very satisfactory in detecting leaks.

The National Tube Company has recommended cast steel bends with flanged ends for the reason that these can be tested easily and the flanged joint permits greater latitude in erection.

Weld Test

A weld test should be made on test pieces cut from the welded portion of a certain percentage of pipe lengths from the make-up of the pipe. The test piece is given both bending and tension tests. The tension test follows standard practice and the value of tensile stress must not be less than 45,000 lb. per square inch. The bending test is made as follows: The test specimen is flattened and ground off so that the weld is of the same thickness as the pipe plate from which the specimen was taken; the specimen is clamped on an anvil with the center of the weld even with the top of the anvil; the projecting end shall then be struck with a sledge, toward the anvil, so that the top of the weld is stretched or elongated; the blows, which should not be too heavy, are continued until the specimen has bent cold through 90 deg.; after bending, the specimen should not show fracture in the weld.

It is very desirable that a means be devised for accurately measuring the thickness at the weld. There is a tendency to hammer the weld too thin and such a device would eliminate this trouble.

Field Tests

It is believed that field pressure tests above normal pressure are desirable, but they are seldom made on account of the expense and difficulty of closing the ends of the pipe. All pipes should, of course, be tested for leakage under normal pressure after erection is complete.

General Tests and Inspection

It is of utmost importance to see that all bump joints are properly fitted in the shop. These should fit so well that nothing but caulking on the outside in the field will be required to make them tight when fitted. Bump joints should be marked with "match marks" and these should be on the top of the pipe when laid.

Certified mill tests of physical and chemical properties of the steel should be furnished by the manufacturer.

The Boiler Code of the American Society of Mechanical Engineers contains some very valuable specifications that are applicable to welded and riveted penstocks. These portions of the Code are printed in the November 1922 issue of *Mechanical Engineering.*

Specifications for Steel

With very few exceptions, general practice has adopted the Standard Specifications of the American Society for Testing Materials. The following specifications are used for the principal parts of penstocks.

Plate for Welded Pipe:

American Society for Testing Materials' *Tentative* Specifications for *Steel for Forge Welding.*

NOTE: There are three important points that should be borne in mind in reference to the strength of steel for welded pipe: first, and most important, elastic limit; second, tensile strength; and third, ductility. With the advent of special steels very high in tensile strength, as long as designs are based strictly on the tensile strength there is a tendency to encourage manufacturers to increase the same in order to save weight on the design. This tensile strength is generally increased with a decided loss in ductility, which is bad for pipe-line design. Generally speaking, it is believed that the tensile strength should be reasonably low to allow for good ductility, and the elastic limit should be made as high as possible with this comparatively low tensile strength. It is also believed that this type of pipe would be more nearly able to withstand water hammer and general abuse than a brittle pipe of high tensile strength. It must be borne in mind, however, that a premium must be paid for any steel that is different from the standards already adopted by the steel manufacturers.

Plate for Riveted Pipe:

American Society for Testing Materials *Flange Steel.*

Rivets:

American Society for Testing Materials *Boiler Rivet Steel.*

Steel Castings:

American Society for Testing Materials *Class B Steel Castings, Medium and Soft Grades* (Soft Grade preferred).

NOTE: It is considered permissible to weld moderate shrink cracks or defects in cast steel pipe fittings. All pipe fittings should be hydrostatically tested at the point of manufacture to a pressure sufficiently above operating conditions to make it plain that there are no defects. In all cases the test pressure should not be less than 50% above the working pressure of the fitting. Weld spots should be thoroughly hammer tested while the casting is under pressure.

For convenience, copies of the principal parts of these specifications are appended.

TENTATIVE SPECIFICATIONS FOR STEEL PLATES FOR FORGE WELDING AMERICAN SOCIETY FOR TESTING MATERIALS

Serial Designation: A 78-21 T

This is a *Tentative Standard* only, published for the purpose of eliciting criticism and suggestions. It is not a Standard of the Society and until its adoption as Standard it is subject to revision.

Issued, 1919; Revised, 1920, 1921.

Material Covered. 1. These specifications apply to steel plates for forge welding for tank cars and for similar construction.

I. MANUFACTURE

Process. 2. The steel shall be made by the open-hearth process.

II. CHEMICAL PROPERTIES AND TESTS

Chemical Composition. 3. (a) The steel shall conform to the following requirements as to chemical composition:

Carbon	for plates ¾ in. or under in thickness	not over 0.18%
	for plates over ¾ in. in thickness	not over 0.20%
Manganese		0.40–0.60%
Phosphorus		not over 0.04%
Sulfur		not over 0.05%

(b) The composition of steel for forge-welding plates should preferably be free from silicon, nickel or chromium. Where these elements are present the maximum quantity of any one shall not exceed 0.05%.

Ladle Analyses. 4. An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus and sulfur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 3.

Check Analyses. 5. An analysis may be made by the purchaser from a broken tension test specimen representing each melt. The chemical composition thus determined shall conform to the requirements specified in Section 3.

III. PHYSICAL PROPERTIES AND TESTS

Tension Tests. 6. (a) The material shall conform to the following minimum requirements as to tensile properties:

Tensile strength, lb. per sq. in.	50,000
Yield point, lb. per sq. in.	0.5 tens. str.
Elongation in 8 in., per cent	$\frac{1{,}500{,}000}{\text{Tens. str.}}$

(b) The yield point shall be determined by the drop of the beam of the testing machine.

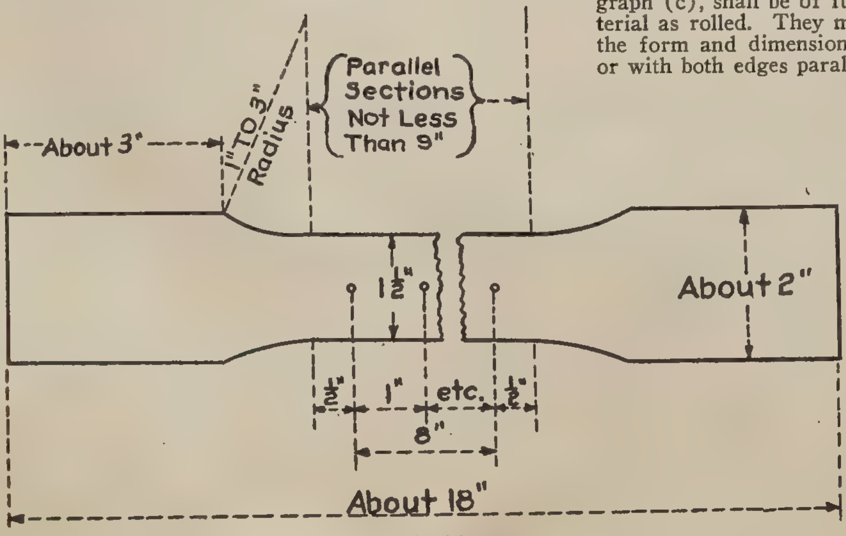
Modifications in Elongation. 7. (a) For material over ¾ in. in thickness, a deduction from the percentage of elongation specified in Section 6 (a) of 0.25% shall be made for each increase of 1/32 in. of the specified thickness above ¾ in. to a minimum of 20%.

(b) For material under 5/16 in. in thickness a deduction from the percentage of elongation in 8 in. specified in Section 6 (a) of 1.25% shall be made for each decrease of 1/32 in. of the specified thickness below 5/16 in.

Bend Tests. 8. The test specimen shall bend cold through 180 deg. flat on itself without cracking on the outside of the bent portion.

Test Specimens. 9. (a) Test specimens shall be prepared for testing from the material in its rolled condition.

(b) Test specimens shall be taken longitudinally and, except as specified in Paragraph (c), shall be of full thickness of material as rolled. They may be machined to the form and dimensions shown in Fig. 1, or with both edges parallel.



(FIG. 1.)

(c) Test specimens for plates over 1½ in. in thickness may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

(d) The machined sides of rectangular bend test specimens may have the corners rounded to a radius not over 1/16 in.

Number of Tests.

10. (a) One tension and one bend test shall be made from each melt, except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(b) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(c) If the percentage of elongation of any tension test specimen is less than that specified in Section 6 (a) and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

IV. PERMISSIBLE VARIATIONS IN WEIGHT AND THICKNESS

Permissible Variations.

11. The cross-section or weight of plates shall be covered by the following permissible variations. One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

(a) *When Ordered to Weight per Square Foot:* The weight of each lot[1] in each shipment shall not vary from the weight ordered more than the amount given in Table I.

1. The term "lot" applied to Table I means all of the plates of each group width and group weight.
2. The term "lot" applied to Table II means all of the plates of each group width and group thickness.

(b) *When Ordered to Thickness:* The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot[2] in each shipment shall not exceed the amount given in Table II.

V. FINISH

Finish.

12. The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI. MARKING

13. The name or brand of the manufacturer and the melt number shall be legibly rolled or stamped on all finished material. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII. INSPECTION AND REJECTION

14. The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

Rejection.

15. (a) Unless otherwise specified, any rejection based on tests made in accordance with Section 5 shall be reported within five working days from the receipt of samples.

(b) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

Rehearing.

16. Samples tested in accordance with Section 5, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

STANDARD SPECIFICATIONS FOR BOILER AND FIREBOX STEEL FOR LOCOMOTIVES

Serial Designation: A 30-21

These specifications are issued under the fixed designation A 30; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision.

Adopted, 1901; Revised, 1909, 1912, 1913, 1914, 1916, 1918, 1921.

Material Covered.

1. These specifications cover two classes of steel for boilers for locomotives, namely; flange and firebox.

I. MANUFACTURE

Process.

2. The steel shall be made by the open-hearth process.

II. CHEMICAL PROPERTIES AND TESTS

Chemical Composition.

3. The steel shall conform to the following requirements as to chemical composition:

	Flange	Firebox
Carbon for plates ¾ in. or under in thickness	......	0.12–0.25%
Carbon: for plates over ¾ in. in thickness....	......	0.12–0.30%
Manganese: for plates ¾ in. or under in thickness	0.30–0.60	0.30–0.50%
Manganese: for plates over ¾ in. in thickness ...	0.30–0.60	0.30–0.60%
Phosphorus:		
Acid	not over 0.05	not over 0.04%
Basic	not over 0.04	not over 0.035%
Sulfur	not over 0.05	not over 0.04%

Ladle Analyses.

4. An analysis of each melt of steel shall be made by the manufacturer to determine the percentage of the elements specified in Section 3. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 3.

Check Analyses.

5. An analysis may be made by the purchaser from a broken tension test specimen representing each plate as rolled. The

chemical composition thus determined shall conform to the requirements specified in Section 3.

III. PHYSICAL PROPERTIES AND TESTS

Tension Tests.

6. (a) The material shall conform to the following requirements as to tensile properties:

	Flange	Firebox
Tensile Strength lb. per sq. in...	55 000-65 000	52 000-62 000
Yield point, min. lb. per sq. in...	0.5 tens. str.	0.5 tens. str.
Elongation in 8 in.	1 500 000	
min. per cent...	1 500 000 / Tens. Str.	1 500 000 / Tens. Str.
(See Section 7)		

(b) The yield point shall be determined by the drop of the beam of the testing machine.

Modifications in Elongation.

7. (a) For material over ¾ in. in thickness, a deduction from the percentages of elongation specified in Section 6 (a) of 0.125 per cent shall be made for each increase of 1/32 in. of the specified thickness above ¾ in.

(b) For material ¼ in. or under in thickness, the elongation shall be measured on a gage length of 24 times the thickness of the specimen.

Bend Tests.

8. The test specimen shall bend cold through 180 deg. without cracking on the outside of the bent portion, as follows: For material 1 in. or under in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over 1 in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

Homogeneity Tests.

9. For firebox steel, a sample taken from a broken tension test specimen shall not show any single seam or cavity more than ¼ in. long, in either of the three fractures obtained in the test for homogeneity, which shall be made as follows:

The specimen shall be either nicked with a chisel or grooved on a machine, transversely, about 1/16 in. deep, in three places about 2 in. apart. The first groove shall be made 2 in. from the square end; each succeeding groove shall be made on the opposite side from the preceding one. The specimen shall then be firmly held in a vise, with the first groove about ¼ in. above the jaws, and the projecting end broken off by light blows of a hammer, the bending being away from the groove. The specimen shall be broken at the other two grooves in the same manner. The object of this test is to open and render visible to the eye any seams due to failure to weld up or to interposed foreign matter, or any cavities due to gas bubbles in the ingot. One side of each fracture shall be examined and the lengths of the seams and cavities determined, a pocket lens being used if necessary.

Test Specimens.

10. (a) Tension test specimens shall be taken longitudinally from the bottom of the finished rolled material, and bend test specimens shall be taken transversely from the middle of the top of the finished rolled material. The longitudinal test specimens shall be taken in the direction of the longitudinal axis of the ingot, and the transverse test specimens at right angles to that axis.

(b) Tension and bend test specimens shall be of the full thickness of material as rolled, and shall be machined to the form and dimensions shown in Fig 1*, except that bend test specimens may be machined with both edges parallel.

Number of Tests.

11. (a) One tension and one bend test shall be made from each plate as rolled.

(b) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(c) If the percentage of elongation of any tension test specimen is less than that specified in Section 6 (a) and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

*For sketch of test specimen see Specifications for Steel for Forge Welding.

V. FINISH

Finish.

13. The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI. MARKING

Marking.

14. (a) The name or brand of the manufacturer, melt or slab number, class, and lowest tensile strength for its class specified in Section 6 (a), shall be legibly stamped on each plate. The melt or slab number shall be legibly stamped on each test specimen.

(b) When specified on the order, plates shall be match-marked, as defined in paragraph (c) so that the test specimens representing them may be identified. When more than one plate is sheared from a single slab or ingot, each shall be match-marked so that they may all be identified with the test specimen representing them.

(c) Each match-mark shall consist of two overlapping circles each not less than 1½ in. in diameter, placed upon the shear lines, and made by separate impressions of a single-circle steel die.

(d) Match-marked coupons shall match with the sheets represented, and only those which match properly shall be accepted.

Inspection, Rejection and Rehearing

For these clauses see Specifications for Steel for Forge Welding.

STANDARD SPECIFICATIONS FOR BOILER RIVET STEEL

Serial Designation: A 31-21

These specifications are issued under the fixed designation A 31; the final number indicates the year of original adoption as standard, or, in the case of revision, the year of last revision.

Adopted, 1901; Revised, 1909, 1912, 1913, 1914, 1921.

A. *REQUIREMENTS FOR ROLLED BARS*

I. MANUFACTURE

Process.

1. The steel shall be made by the open-hearth process.

II. Chemical Properties and Tests

Chemical Composition. 2. The steel shall conform to the following requirements as to chemical composition:

Manganese 0.30–0.50%
Phosphorus ....not over 0.04%
Sulfur .... not over 0.45%

Ladle Analyses. 3. An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus and sulfur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 2.

Check Analyses. 4. Analyses may be made by the purchaser from finished bars representing each melt. The chemical composition thus determined shall conform to the requirements specified in Section 2.

III. Physical Properties and Tests

Tension Tests. 5. (a) The bars shall conform to the following requirements as to tensile properties:

Tensile strength, lb. per sq. in. 45,000–55,000
Yield point, min. lb. per sq. in. 0.5 ten. str.
Elongation in 8 in., min., per cent .... $\frac{1{,}500{,}000}{\text{ten. str.}}$
but need not exceed 30%

(b) The yield point shall be determined by the drop of the beam of the testing machine.

Bend Tests. 6. (a) *Cold-Bend Tests:* The test specimen shall bend cold through 180 deg. flat on itself without cracking on the outside of the bent portion.

(b) *Quench-Bend Tests:* The test specimen, when heated to a light cherry red as seen in the dark (not less than 1200 deg. Fahr.), and quenched at once in water the temperature of which is between 80 and 90 deg. Fahr., shall bend through 180 deg. flat on itself without cracking on the outside of the bent portion.

Test Specimens. 7. (a) Test specimens shall be of the full diameter of bars as rolled.

(b) Tension and bend test specimens for rivet bars which have been cold drawn shall be normalized before testing.

Number of Tests. 8. (a) Two tension, two cold-bend and two quench-bend tests shall be made from each melt, each of which shall conform to the requirements specified.

(b) If any test specimen develops flaws it may be discarded and another specimen substituted.

(c) If the percentage of elongation of any tension test specimen is less than that specified in Section 5 (a) and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

IV. Permissible Variations in Diameter

Permissible Variations. 9. The diameter of each bar shall not vary more than 0.01 in. from that specified.

V. Workmanship and Finish

Workmanship. 10. The finished bars shall be circular within 0.01 in.

Finish. 11. The finished bars shall be free from injurious defects and shall have a workmanlike finish.

VI. Marking

Marking. 12. Rivet bars shall, when loaded for shipment, be properly separated and marked with the name or brand of the manufacturer and the melt number for identification. The melt number shall be legibly marked on each test specimen.

Inspection, Rejection and Rehearing

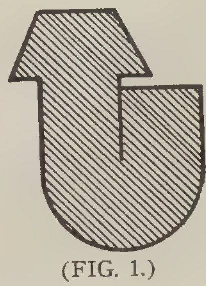
For these clauses see Specifications for Steel for Forge Welding.

B. *REQUIREMENTS FOR RIVETS*

VIII. Physical Properties and Tests

Tension Tests. 16. The rivets, when tested, shall conform to the requirements as to tensile properties specified in Section 5, except that the elongation shall be measured on a gage length not less than four times the diameter of the rivet.

17. The rivet shank shall bend cold through 180 deg. flat on itself, as shown in Fig. 1, without cracking on the outside of the bent portion.



(FIG. 1.)

(FIG. 2.)

Flattening Tests. 18. The rivet head shall flatten, while hot, to a diameter $2\frac{1}{2}$ times the diameter of the shank, as shown in Fig. 2, without cracking at the edges.

Number of Tests. 19. (a) When specified, one tension test shall be made from each size in each lot of rivets offered for inspection.

(b) Three bend and three flattening tests shall be made from each size in each lot of rivets offered for inspection, each of which shall conform to the requirements specified.

IX. Workmanship and Finish

Workmanship. 20. The rivets shall be true to form, concentric, and shall be made in a workmanlike manner.

Finish. 21. The finished rivets shall be free from injurious defects.

STANDARD SPECIFICATIONS FOR STEEL CASTINGS SERIAL DESIGNATION: A 27-21

These specifications are issued under the fixed designation A 27; the final number indicates the year of original adoption as standard, or, in the cases of revision, the year of last revision.

Adopted, 1901; Revised, 1905, 1912, 1913, 1914, 1916, 1921.

Material Covered. 1. These specifications cover two classes of castings, namely:

Class A, ordinary castings for which no physical requirements are specified;

Class B, castings for which physical requirements are specified. These are of three grades: hard, medium and soft.

Patterns. 2. (a) Patterns shall be made so that sufficient finish is allowed to provide for all variations in shrinkage.

(b) Patterns shall be painted three colors to represent metal, cores and finished surfaces. It is recommended that core prints shall be painted black and finished surfaces red.

Basis of Purchase. 3. The purchaser shall indicate his intention to substitute the test to destruction specified in Section 11 for the tension and bend tests, and shall designate patterns from which castings for this test shall be made.

I. MANUFACTURE

Process. 4. The steel shall be made by one or more of the following processes: open-hearth, electric furnace, side blow converter or crucible.

Heat Treatment. 5. (a) Class A castings need not be annealed unless so specified.

(b) Class B castings shall be properly annealed, the treatment depending upon the design and chemical composition of the castings.

II. CHEMICAL PROPERTIES AND TESTS

Chemical Compositions. 6. The castings shall conform to the following requirements as to chemical composition:

	Class A	Class B
Carbon ...	not over 0.30%	....
Phosphorus		
Acid ...	not over 0.07%	not over 0.06%
Basic ..	not over 0.06%	not over 0.05%
Sulfur.....	....	not over 0.06%

Ladle Analyses. 7. An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus and sulfur. This analysis shall be made from drillings taken at least ¼ in. beneath the surface of a test ingot obtained during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 6.

Check Analyses. 8. (a) Analyses of Class A castings may be made by the purchaser. The phosphorus content thus determined shall not exceed that specified in Section 6 by more than 20%. Drillings for analysis shall be taken not less than ¼ inch beneath the surface.

(b) Analyses of Class B castings may be made by the purchaser from a broken tension or bend test specimen. The phosphorus and sulfur content thus determined shall not exceed that specified in Section 6 by more than 20%. Drillings for analysis shall be taken not less than ¼ in. beneath the surface.

III. PHYSICAL PROPERTIES AND TESTS
(For Class B Castings Only)

Tension Tests. 9. (a) The castings shall conform to the following minimum requirements as to tensile properties:

	Hard	Medium	Soft
Tensile str., lb. per sq. in.	80 000	70 000	60 000
Yield point, lb. per sq. in.	0.45 t. s.	0.45 t. s.	0.45 t. s.
Elongation in 2 in. per cent.	15	18	22
Reduction of area, per cent	20	25	30

(b) The yield point shall be determined by the drop of the beam of the testing machine.

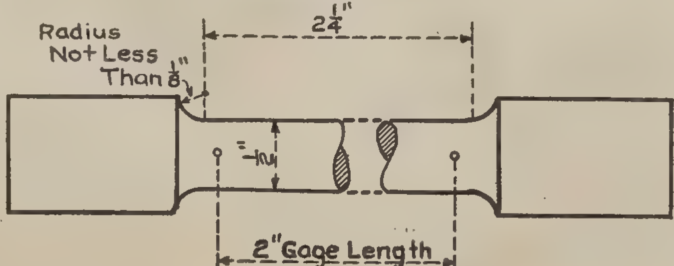
Bend Tests. 10. (a) The test specimen for soft castings shall bend cold through 120 deg., and for medium castings through 90 deg., around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(b) Hard castings shall not be subject to bend test requirements.

Alternative Tests to Destruction 11. In the case of small or unimportant castings, a test to destruction on three castings from a lot may, upon agreement between the manufacturer and the purchaser, be substituted for the tension and bend tests. This test shall show the material to be ductile, free from injurious defects, and suitable for the purpose intended. Unless otherwise agreed upon between the manufacturer and the purchaser, a lot shall consist of all castings from one melt, in the same annealing charge.

Test Specimens. 12. (a) Sufficient test bars, from which the test specimens required in Section 13 (a) may be selected, shall be attached to castings weighing 500 lb. or over, when the design of the casting will permit. If the castings weigh less than 500 lb., or are of such a design that test bars cannot be attached, two test bars shall be cast to represent each melt; or the quality of the castings shall be determined by tests to destruction as specified in Section II. All test bars shall be annealed with castings they represent.

(b) The manufacturer and purchaser shall agree whether test bars can be attached to castings, on the location of the bars on the castings, on the castings to which bars are to be attached, and on the method of casting unattached bars.



(FIG. 1)

(c) Tension test specimens shall conform to the dimensions shown in Fig. 1. The ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens shall be machined to 1 by ½ in. in section with corners rounded to a radius not over $\frac{1}{16}$ in.

Number of Tests. 13. (a) One tension and one bend test shall be made from each annealing charge. If more than one melt is represented in an annealing charge, one tension and one bend test shall be made from each melt.

(b) If any test specimen shows defective machining or develops flaws, it may be discarded; in which case the manufacturer and the purchaser or his representative shall agree upon the selection of another specimen in its stead.

(c) If the percentage of elongation of any tension test specimen is less than that specified in Section 9 (a) and any part of the fracture is more than ¾ in. from the center of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

Retests. 14. If the results of the physical tests of any test lot do not conform to the requirements specified, the manufacturer may reanneal such lot not more than twice and retests shall be as specified in Sections 9 and 10.

IV. Workmanship and Finish

Workmanship. 15. The castings shall substantially conform to the sizes and shapes of the patterns, and shall be made in a workmanlike manner.

Finish. 16. (a) The castings shall be free from injurious defects.

(b) Minor defects which do not impair the strength of the castings may, with the approval of the purchaser or his representative, be welded by an approved process. The defects shall first be cleaned out to solid metal; and after welding, the castings shall be annealed, if specified by the purchaser or his representative.

(c) Castings shall not be offered for inspection if covered with paint, rust, or any other substance to such an extent as to hide defects.

Inspection, Rejection and Rehearing

For these clauses see Specifications for Steel for Forge Welding.

MANUFACTURERS' STATEMENTS

Statement By American Spiral Pipe Works

It would appear that 90 per cent is safe for the efficiency of the weld when using 50,000 to 60,000 tensile strength steel. Our test shows the average way beyond this amount.

The type of girth joints is dependent very largely on conditions. The larger sizes naturally are more economical with the field riveted joints in which the ends of the pipe are flared and tapered so they are readily inserted together. However, this requires considerable work in the field.

In the smaller sizes it is to be found more preferable to have the joints made up complete at the place of manufacture.

Our high pressure bolted socket joints are very satisfactory for this work and, for extremely high pressures, the follower ring type is very satisfactory.

We suggest that smaller sizes of pipe, possibly up to 48 in., be made of sufficient thickness to stand a vacuum, as the small amount invested in this material is not a big item compared with the safety.

It is customary for us to use the best hemp packing for expansion joints after resoaking this in hot mutton tallow, as we believe this adds materially to its life and performance.

Bends are naturally made in four ways. Sections of lap welded pipe are cut and girth seams riveted on, or forge welded, or made from lap welded pipe, which is cut *two-thirds* of the distance around and the remainder left solid. They may also be bent from a single piece of pipe in the usual manner, diameters and pressures governing very largely as to which method would be most economical.

Customary shop practice is 50 per cent in excess of static head. The thickness of the weld can be ascertained by a pair of long calipers.

If the girth seams are designed of sufficient strength, there is no doubt but a span of 40 ft. can be made with safety.

It is possible something better than plain concrete could be worked out for supporting the pipe. However, it is customary to figure a friction of 20 per cent over the piers.

The nozzles should be seamless forged steel. Round rubber gaskets crushed down to a thin film, make a very satisfactory joint for high pressure.

Our patented bolted socket joint has proven most satisfactory in smaller lines where the pipe follows the contour of the ground and makes the required bends, as this joint is prevented from pulling apart by the bead on the end of the pipe, and, in the smaller sizes, can stand blanking off with full pressure, which means that no anchoring is required when making curves in the line, and is tight under the highest pressure.

We are prepared to furnish this pipe in sizes up to 96 in. and thickness up to 1½ in.

Statement By M. W. Kellogg Company

Limiting Sizes and Equipment

We are prepared to make and test lap welded pipe from 24 in. internal diameter to any diameter which can be shipped. At the present time we find that we can ship material 11 ft.-0 in. internal diameter to the Pacific Coast over certain routings. Our equipment at the present time is sufficient to weld all material from ¼ in. in thickness up to and including in. in thickness. We are equipped for lengths as follows: Our rolls will make pipe of a nominal overall length of 20 ft., which, if the bump joint construction is used, will lay from 19 ft.-3 in. to 19 ft.-6 in., depending on diameters, thicknesses, etc. We are also equipped to make pipes in nominal lengths of 30 ft.-0 in., which will lay about 29 ft.-4 in. to 29 ft.-6 in.

We, of course, can make longer lengths by end-welding pipes together, and we can make intermediate lengths by welding pipes together, but the two lengths given above are the most economical to handle with our present equipment. We can also make all the reducers necessary. In the case of a reducer we would make the actual reduction in a length of not over 10 ft. by welding a 5 ft. cylindrical section on each end of the 10 ft. reducer to bring it up to the nominal length of 20 ft. This is necessary so as to get the proper bump joints on the cylindrical sections. All our material is tested in a special hydraulic testing machine, which will take material up to 11 ft.-6 in. internal diameter, and exert an end pressure to offset internal pressure of 2,000 tons. We recommend that pieces entering into the construction of a pipe line be given a hydraulic test to at least one and one-half times the pressure it is to withstand in the profile, and that they be vigorously hammered with a ten-pound sledge while

under pressure; and that special apparatus be used to simulate water hammer or shock as much as possible.

Joints

We have found from actual field experience that it is of the utmost importance that all bump joints be individually fitted in the shop and drilled in position. Our procedure is to rough roll the joints, expand the female end so that the male end will enter properly, place the combined pipes in the furnace and then, by means of a power hammer, fit the female carefully to the male, after which, the pipes being held together by reach rods, are placed in the drilling machine, and drilled in position. The pipes are carefully marked for assembly in the field.

We would like to particularly draw attention to the fact that with the modern type of construction on pipe lines, using intermediate expansion joints between the anchors, the circumferential stresses are very much reduced, thus allowing a very much smaller rivet to be used for driving in the field. This is also true when the joint is carefully fitted in the factory, as the rivets are then not required to pull the heavy material metal to metal. In this way, even on very heavy material, we can keep the field riveting down to a diameter of 1¼ in. or less, with the large majority of cases 1 in. or less. We never use a rivet larger than 1¼ in.

Rivets

We have found from actual experience that the cone type of rivet seems to give the best field job and requires less skill in driving than the button type. We are aware that in this country practically all rivets are passed into the hole from the inside of the pipe and driven from the outside. The writer, however, has seen in India, where very unskilled labor is used for this type of work, that very good results have been achieved by driving the rivet from the inside and holding on from the outside. In this case a steel strap is put around the pipe to hold the outside gun, two guns being used, one inside and one outside. Whereas we are not prepared to say whether this is the better method of driving, but we believe that with unskilled labor a better job can be done by driving from the inside. This also eliminates the necessity for pass holes, although we have found that pass holes are very convenient for lighting and ventilating the inside of the pipes, particularly in hot weather, and they are also very convenient for passing in air hose, etc. For all work in the United States we have furnished pass holes either in every length or in every other length of pipe. As far as possible we try to keep the welding line on the top center of the pipes and place the pass holes about 15 deg. either side of the top center. This is particularly true where one or more lines are placed parallel, so that if anything should happen to a seam, the direction of the water would be upwards. On double welded material we generally place the welds on the sides of the pipes.

Tests made by us on the weld itself have proven that the elastic limit is affected very little by the welding, and as we guarantee an efficiency of 90 per cent of the strength of the pipe based on the tensile strength, it is safe to take the elastic limit through the weld as practically 90 per cent of the elastic limit through the plate.

Minimum Thickness

The minimum thickness of pipe, of course, is governed by a number of points other than the weldability of the material, such as collapsing, handling in the field, etc. We believe from actual experience that pipe to be used in field erection in diameters around 5 ft. should not be made of material less than ½ in. thick. We find that material ⅜ in. and $\frac{7}{16}$ in. is often damaged in shipping, the joints being very much distorted, and we believe that more money is spent in the long run than would be the case if the pipe were of proper thickness. It is also without doubt that the welding is more reliable on pipe ½ in. and over in thickness, due to the fact that the hammers and special apparatus are built for heavy material. Whereas, with care, a good job can be made with lighter material, from a practical standpoint, the cost of the lighter material is increased, which might offset any saving in weight. We feel, therefore, that for diameters of 5 ft. and over not less than ½ in. thick material should be used, and for diameters less than 5 ft. material not less than $\frac{7}{16}$ in. thick should be used, although we are prepared to furnish lighter material if required.

Expansion Joints Equipped with Sleeves

Up to the present time we have been furnishing expansion joints of the regular type without any special protective coating for the male or sleeve member. We have realized for some time past that there are certain objections to this procedure. The actual motion of the pipes when filled with water is very small, consequently the packing is very apt to stay in one position for a considerable length of time; what seepage there is through the packing has a tendency to pit or corrode the pipe at this particular point. This is more or less serious, depending on local conditions. After the pipe is emptied and the expansion joint gets considerable travel, the pitted portion of the pipe is apt to tear the packing. To overcome this matter, we have developed and patented a special copper sleeve electro-plated on to the steel member. We were confronted with the difficulty of placing a brass or bronze liner on the pipe, particularly of large diameter, where the cost of attempting to machine the pipe would have been excessive; we also had the difficulty of properly attaching this liner and making it water tight.

We have just completed and installed a complete copper-plating outfit for copper plating this sleeve to the pipe. We get an absolute bond between the copper and the steel, and by the use of proper chemicals, etc., get a very hard and ductile copper deposit. We copper plate these pipes for about two feet from the entering end and have the end of the pipe copper plated as well. After the plating is done the copper is buffed up with an emery wheel, forming a very smooth and bright surface for sliding through the packing. We believe that this will overcome all corrosion and, accordingly, add to the life of the packing and will make it easier to make the joints perfectly tight. We might state that the cost of placing the sleeves on the pipe in this manner is very moderate in comparison with the result obtained. We have found it inadvisable to make this sleeve too thick; a copper plating of less than $\frac{1}{32}$ in. of the proper consistency is more than ample to withstand the abuse and give the proper results.

Laterals and "Y" Pieces

The advent of the double overhung impulse wheel, together with the ever-increasing heads, have made it economical and desirable to break a pipe into two lines at some pre-determined point in the pipe line. This is particularly true where diameters and thicknesses are such that mechanical difficulties are involved in making pipes of greater thicknesses. It is, however, desirable that this bisecting point should be placed down the profile as far as possible so as to take advantage of the heavy material. The problem of breaking one pipe into two under any considerable pressure has always been a more or less difficult one and has been solved in the past by the use of cast steel. It has been found difficult to make steel castings of this character free from imperfections, and it is also a problem to take the unbalanced forces originating from a cross-section of a non-circular section and balance same. We have therefore evolved a special lateral or "Y" piece, patented by us, which we believe very successfully overcomes these difficulties. We might state that we have a number of these "Y" pieces in operation, and are enclosing several photographs.

We have 68-in. pipe breaking into two 46-in. pipes, and 42-in. pipe breaking into smaller pipes, etc., under heads as high as 1900 ft. In this case we are breaking from a 42-in. pipe into two 34-in. pipes; the 34-in. pipes are made

first in a conical section having a circumference at the upstream end equal to one-half the circumference of the 42-in. pipe, plus one diameter, plus a small additional amount of circumference to take care of fillets, etc. After these sections are made up and tested, they are reheated and placed in a special die in a powerful hydraulic press and the semi-elliptical flat sections as shown on the sketch are pressed into the pipe. The two pipes are then placed together with the flattened sections coinciding. In this way at the 42-in. pipe end we have a slight elliptical section having a circumference equal to the 42-in. pipe, and the 42-in. pipe is then formed to coincide with this section and the two are welded together. Before this welding is done, however, the two flat sections are welded together *at their upstream end only.*

After this forge welding has been done—joining the 42-in. pipe to the two 34-in. pipes, as described above—seamless steel bands are properly formed and placed around the two 34-in. pipes, as shown; tapered wedges are then carefully driven in between the seamless steel bands and the perimeter of the pipes, thus evenly distributing the load. They are thereafter welded in place to firmly secure them. It will be seen that the unbalanced forces are taken care of as follows: The elliptical section of the 42-in. pipe is kept in place by the two flat surfaces acting as a diaphragm. It will also be noted that the unbalanced forces are simply the pressure acting upon the semi-elliptical section of the flattened pipes. These forces are taken in direct tension by the seamless bands, thus allowing us to build this lateral for any heads or pressures. It should also be noted that the flow lines are extremely easy and very little head is lost in the lateral itself. The diaphragm on the extreme end is sometimes cut in a circular section to increase the respective areas at this point. This lateral can be made with any included angle up to 45 deg.

We have had very good results with angles of around 30 to 35 deg., as these angles seem to make easy flow lines possible. It is also possible to make this lateral in very large sizes for assembly in the field, in which case the upstream single pipe is attached to the two downstream pipes by butt straps and rivets, and the seamless bands are replaced by forming sheet steel members which are riveted to the downstream legs, thus taking the tension. It is also possible on very large sizes to substitute for the seamless band or sheet steel members reinforced concrete. Consequently the size of this lateral is only limited to the ability to ship any one of its three members.

A lateral of this type allows us to make full use of our ability to make pipe up to 2 in. thick, after which the single pipe can be broken into two smaller ones, running them, if necessary, to where the material is 2 in. thick, and breaking again. This construction incidentally saves a great deal of space on the profile, thus saving costly excavation, etc. It also saves in regulating valves and other devices. We have used this construction on Southern California Edison Company Big Creek No. 2 and Big Creek No. 1 pipe lines, and the Braden Copper pipe lines at Rangagua, Chile, under heads of 1700 ft. and are at present making three of a similar nature for the Shinyetsu Electric Power Co., Tokyo, Japan.

STATEMENT BY NATIONAL TUBE COMPANY

Buried vs. Exposed Pipe

We believe that exposed pipe for hydro-electric installation is a far better proposition, as this permits periodical inspection and accessibility to parts that require attention.

Welded Pipe

As to the girth joints, the bump joints are, of course, the most desirable. It is our belief that at a certain wall thickness, say 1⅛ in. to 1¼ in., the bump joint should be eliminated and some other type of joint substituted. However, there is a tendency of late, especially among western concerns, to go to the extreme on heavy wall with the use of bump joints. It is our practice to put the pass holes 18 in. from the end of male joint in line with master hole, i. e., always on top of pipe. On smaller lines we think a pass hole should be placed in each pipe; in larger pipe about every second or third pipe.

We recommend that the minimum thickness be ¼ in. up to 40 in. in diameter, ⅜ in. up to 60 in. and ½ in. up to 96 in.

Minimum Thickness of Steel

It is our belief that it is more desirable to make a large pipe of heavier wall instead of inserting stiffening rings in large pipe of thinner walls. The amount of labor involved scarcely warrants the latter procedure.

Anchor Rings

We believe the anchor rings to be desirable. If they are omitted the whole thrust is concentrated on a small area and the bearing per square foot on concrete is too great. In placing the anchor rings the total thrust pressure is evenly divided and the bearing per unit can be calculated not to exceed the permissible. As to the type, we prefer to use cast iron half rings which are placed around the pipe with a total flange area of such dimensions that the pressure per square inch remains within the permissible limits. The upper rings should be placed in such a way as to butt up against the flange of the bend.

Expansion Joints

We believe an expansion joint to be a very desirable feature. Not considering its duties when placed in this line, we have found it to be a hardship from a manufacturing point of view when no adjustment of the line is provided. The measurement of each pipe with bump joints and bends in a rigid line is very uncertain. We generally measure each item of a complete line two to three times, and invariably two inspectors give us two different sets of dimensions. An expansion joint or any other adjustment of the line will save quite considerable trouble in the process of manufacture. As to the location, we prefer to place it below each foundation and as close as possible. The location of the expansion joint between two foundations has its advantages and disadvantages. The sketch shows a different type of an expansion joint which is more rigid. As to the bronze rings, they may be placed on pipe instead of in the form of glands. It appears to us that a bronze collar forced on pipe by means of hydraulic press will be the best of the two propositions. We do not see exactly why an electrolytically applied copper collar should be especially desirable. We use specially prepared flax packing; this kind of packing has been in service for over a year and we have not heard of any complaints.

Specials

It is our general practice to make the bends up to 22 deg. with one circular seam up to 45 deg. with two seams, up to 90 deg. with four seams. We advocate flanged joints for bends for reasons of manufacturing as well as for field assembly reasons. Somehow the western hydro-electric concerns of late make the bends with bump joints, which is very undesirable from the manufacturing point. Oil testing only should be prohibited on bends, as it is a very undesirable method of testing bends. Special heads should be provided for testing, be it flanged joints or bump joints. Manholes should be forgings riveted or welded to the pipe. The welding of steel casting fittings to the pipe should be prohibited. The same refers to nozzles. A pipe with nozzles and manholes should be tested in assembled condition. When bends or laterals are above 1¼ in. wall it is a question in our minds if a steel casting of sufficient wall thickness should not replace the welded product. Especially on laterals when the weld can be made only by means of hand sledging the dependability of welds of 1¼ in. wall and above are somewhat question-

able. The stiffening rings are scarcely to be taken into consideration, as their worth is very doubtful.

Tests and Inspection

All straight pipe or any specials should be tested to about 50 per cent above the working pressure. If laterals and bends are complicated, special arrangements should be provided for testing. It is an undesirable proposition to test specials with bump joints, as the joint is greatly distorted during the test. The welds should be hammered with an 8-lb. sledge while the pipe is under test. It may be desirable to put the pressure on and take it off in rapid succession. A field test above the working pressure is very desirable. The weld at the end of the pipe is apt to be somewhat thinner than the rest of the weld or than the thickness of the material. We generally provide some additional length and cut off large crop ends. A 10-12% tolerance should be given to manufacturer, although in most of the cases we are able to keep below the above figure.

Piers

It is our general practice to put a small pier under each 20 ft. length of pipe, although we could not give you good reasons for it, simply an old established practice that proved to be satisfactory. This item needs a closer investigation, and we may some time in the future conduct some experiments along these lines. A cast-iron plate or steel plate with grooves for oiling purposes greatly reduces the friction of the pipe on a pier. In Japan and South America they use a heavy paper apparently with satisfactory results. For the purpose of calculation we use the coefficient of friction, 0.50.

Painting

There are many widely diverging opinions concerning the painting of pipe lines. Unless otherwise specified, we use our "National Coating." In the future we intend to install a dipping plant. The process will cover vertical dipping of the pre-heated pipe in a bath of hot compound. We believe this to be the most satisfactory method. In case customer paints his own pipe for *transportation purposes* we use what we call Lorain mix:

Lorain Mix:
75%-90% Wash Benzol
25% Spraying Oil

For very long distances we use Export coating.

Export Coating:
50%-90% Wash Benzol
50% Spraying Oil

N. B.—Spraying oil used is of linseed oil base with china wood oil.

It is our belief that the pipe should be painted inside. The chemical composition of water is often such that deteriorations of the pipe inside are greater than outside.

COMPARATIVE TESTS ON EXPERIMENTAL DRAFT TUBES

The Alabama Power Company has conducted a series of comparative tests on draft tubes at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute, located at Worcester, Mass. (see Fig. 37), to determine a design best suited for its Mitchell Dam hydro-electric plant, and to investigate the merits of the more recent draft tube designs as submitted by the leading water wheel manufacturers of this country, and with the view in mind of using this information in its extensive program of proposed hydro-electric developments.

The problem of obtaining reliable information on the comparative performance of various types of tubes was met by using an actual runner and measuring the overall performance of wheel and tube. The same personnel was used throughout the entire testing program, the only condition changed being the draft tubes.

A 10½-inch bronze runner of the Francis type with a specific speed of 66.7 was used in conducting these experiments, and was an exact model of the large units built for Mitchell Dam by Allis-Chalmers Manufacturing Company of Milwaukee, Wis. All tubes were designed for the same conditions, similar to those of the Mitchell Dam power house, (see N. E. L. A. report of January, 1922) and were proportionately reduced in a ratio of 130 to 10½, which are the respective diameters in inches of the runners. The range of speeds for the model water wheel was 300 to 600 r. p. m., and it developed a maximum of about 14 horsepower under an effective head of 14.5 feet. This gave a maximum discharge of something like 13 cu. ft. per second.

A total of twelve different types of tubes were tested during a period of six months, the final runs having been made in the early part of August, 1922. Most of the changes known to affect the performance of the tubes were tried. Tests were made to show the effect of varying the throat openings and bell diameters, installation of supporting piers in the draft tube bell, cones of various heights, flat plates, effect of extending horizontal diffusers down stream, effect of increasing the height of vertical barrels, effect of enlarging the lower part of the vertical barrel, effect of decreasing areas in quarter turns to better direct the line of flow, turbine without draft tube, and other combinations. An automatic vacuum recorder was built to investigate the vacuum conditions in the various type of tubes. This machine indicated clearly the difficulties of speed regulation in a poor type of tube as compared with a more efficient one. As the performance of

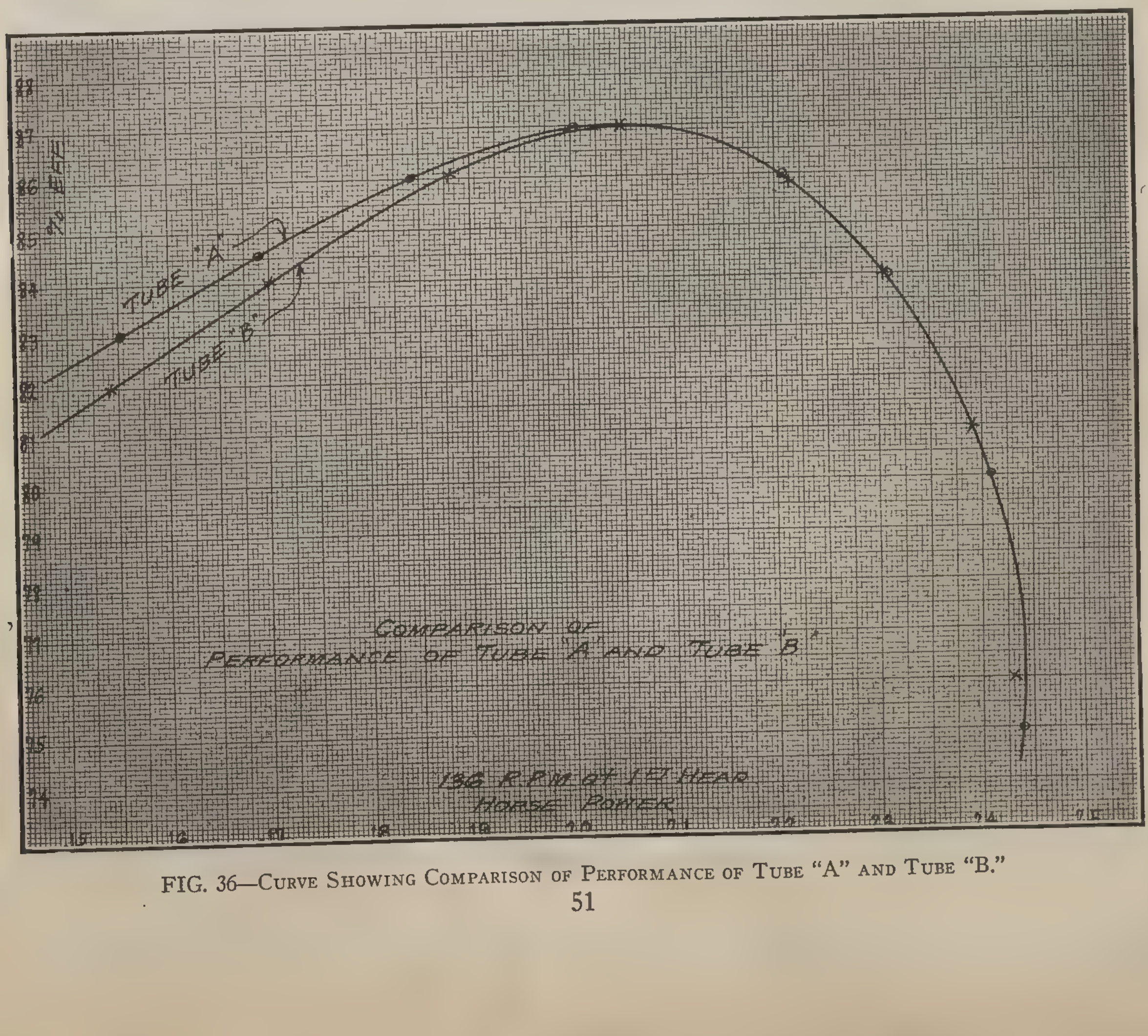


FIG. 36—Curve Showing Comparison of Performance of Tube "A" and Tube "B."

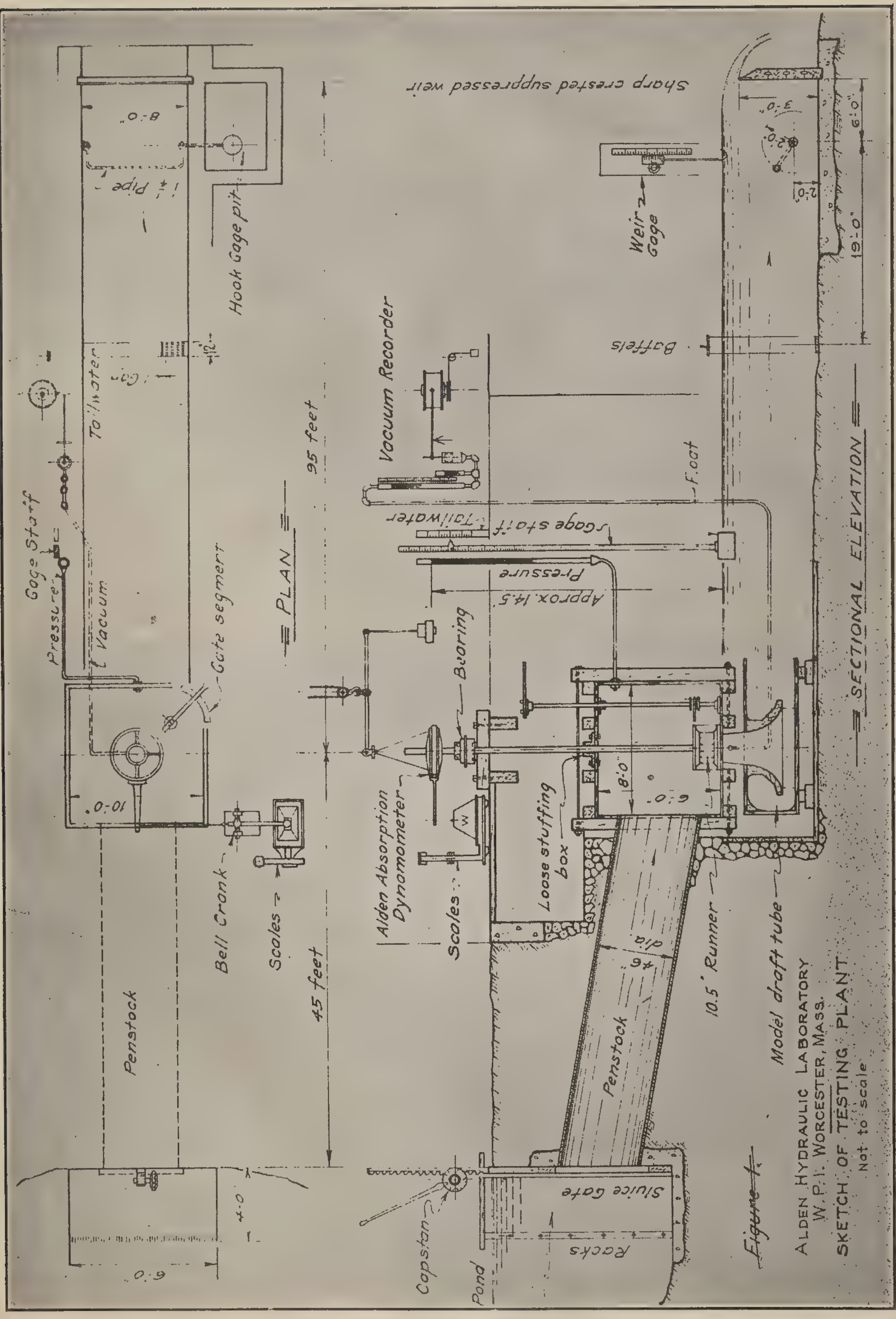


FIG. 37—SKETCH OF TESTING PLANT, ALDEN HYDRAULIC LABORATORY, WORCESTER POLYTECHNIC INSTITUTE, WORCESTER, MASS.

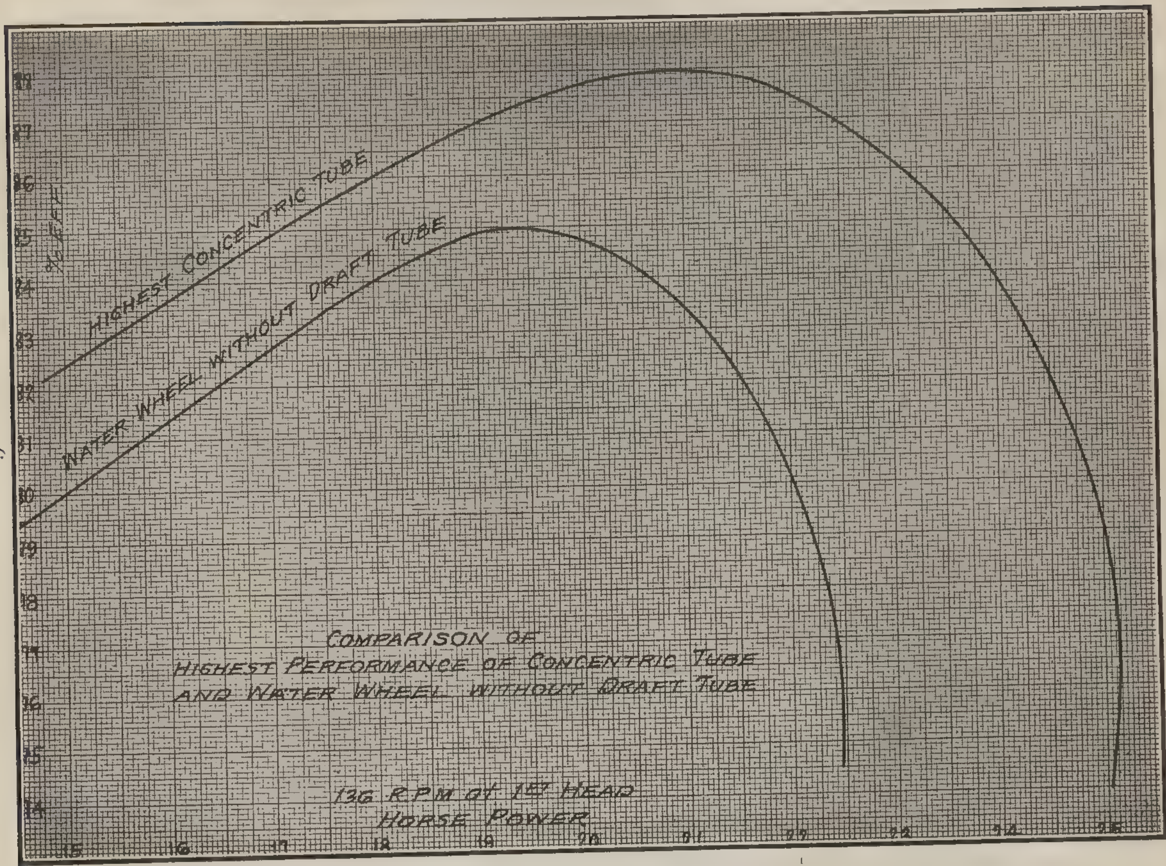


FIG. 38—CURVE SHOWING COMPARISON OF HIGHEST PERFORMANCE OF CONCENTRIC TUBE AND WATER WHEEL WITHOUT DRAFT TUBE.

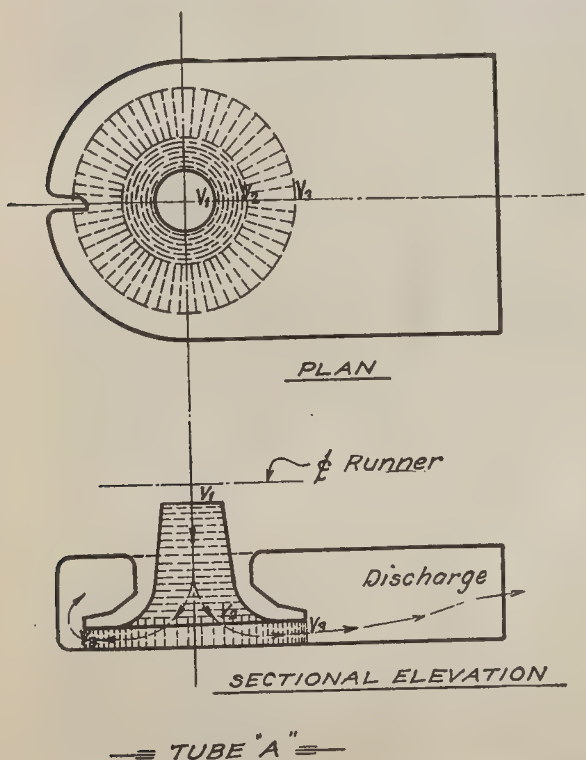


FIG. 39—SKETCH OF DRAFT TUBE "A" FOR MITCHELL DAM—ALABAMA POWER COMPANY.

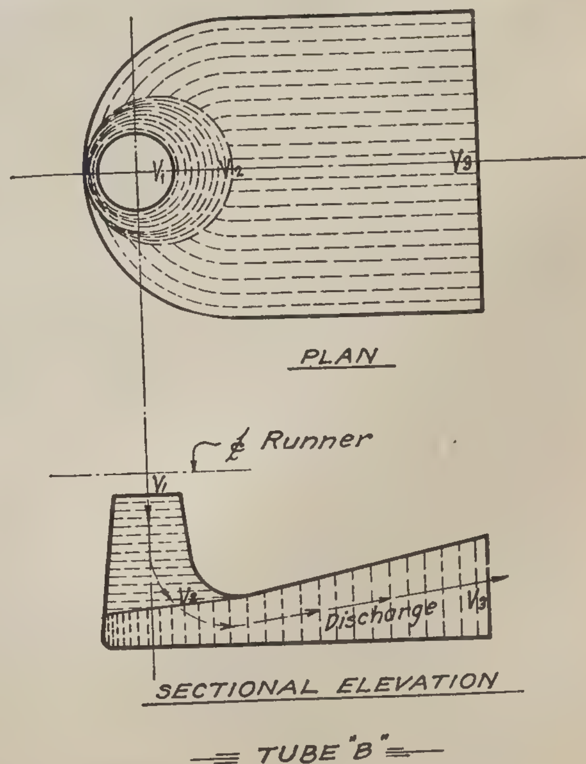


FIG. 40—SKETCH OF DRAFT TUBE "B" FOR MITCHELL DAM—ALABAMA POWER COMPANY

most of the higher efficiency tubes was almost identical, great accuracy was necessary to determine exactly the best conditions. The extent of the closeness of this work can be appreciated when it is considered that repeated check tests to within 1/10 of 1 per cent were made after several changes in the draft tubes and new set-ups in machinery had been made. The difference of over-all maximum efficiency between the lowest and highest condition was 3 per cent. A much higher percentage of increase in maximum horsepower was recorded, being 12 per cent greater with the highest tube against a setting without a draft tube. (See Fig. 38.)

From these tests two designs were selected for Mitchell Dam; one as shown in Fig. 39 was used in units No. 1 and No. 2, and a second as shown in Fig. 40 for No. 3 and No. 4. Hydraulically these two types of tubes performed almost identically. (See Fig. 36).

A complete description of these experiments is to be presented through the American Society of Civil Engineers in 1923.

SALT VELOCITY METHOD OF MEASURING WATER

The salt velocity method of measuring water consists of accurately timing the passage of a charge of brine between two or more known points, which has been injected into the stream flow. By dividing the volume of conduit (cu. ft.) between these points by the time of passage (in seconds) the rate of flow (cu. ft. per sec.) is obtained. The conductivity of water varies directly as its salt content. The brine is usually introduced under air pressure through a piping system designed to give approximately uniform distribution. The timing of the passage of charge is made by means of a stop watch or recording seconds clock and indicating electrical instruments, or recording electrical instruments such as ammeter, voltmeter, wattmeter or watt hour meter.

The time of introduction may be registered by a switch operated in conjunction with a quick opening brine introduction valve or by a pair of electrodes located just below the brine distributing system. The time of passage by the lower points or sections of conduit is obtained by the use of one or more pairs of electrodes, either installed in the conduit or inserted through stuffing boxes. By means of properly designed electrodes, complete traverses of the conduit can be made if desired.

The recording chart can be run at various predetermined speeds to best suit the conditions of test. All events can be registered mechanically or electrically on this chart, such as introduction of brine, passage of brine by one or more sets of electrodes, seconds, revolution of wheel shaft, etc.

From these charts the actual time of brine passage is obtained. So far as investigations have been made the true time is shown, or mean velocity through conduit is given, by the center of gravity of the area recorded on the chart by the charge. In pipe lines of sufficient length the time of the center of gravity of the charge coincides with the time of its maximum conductivity.

This method can be compared with the submerged float method as used in canals of uniform cross section. In the salt velocity method there is almost an infinite number of truly submerged floats instead of a few. With all of the events recorded automatically on the chart, personal errors are eliminated and all electrical calibration of instruments made unnecessary. Comparative tests of this method with the Venturi Meter, weir and weighing tank shows it to be a very accurate means of measuring large or small volumes of water.

FIG. 41—Curve Showing Consistency of Measurements of Discharge by Salt Velocity Method.

Field tests of a water wheel have recently been made where quantities as high as 3500 c.f.s. were measured. The following discharge curve of this wheel shows the consistency of the measurements. The penstock was 20 ft. in diameter and the electrodes were approximately 500 ft. apart.

In one plant where the available length of rectangular penstock was about 55 ft., something over 2,000 individual shots of brine, or tests, were made to thoroughly investigate this method. During these investigations, a remarkable consistency of results was noted. Tests could be repeated day after day and checked.

METHODS OF FORECASTING WATER SUPPLY

Progress Report of Subcommittee

Water Supply

The limiting feature determining the ultimate capacity for a power plant differs according to the kind of power which has to be developed. In almost every case, except where water power is used, the ultimate power required determines the capacity of plant to be installed, but in the case of water power the capacity of the stream, provided it does not exceed the power requirements, determines this capacity.

In too many cases water power developments have been made without a thorough study of the discharge of the stream and its storage possibilities, resulting in an overdevelopment with perpetual excessive overhead expense, or an underdevelopment requiring large capital outlay to bring it up to full development.

The sources from which a determination of this discharge of a stream may be arrived at are very meagre, particularly in the West. The records of the United States Geological Survey will some day in all probability give the information desired, but up to the present time they cannot be accepted without qualification. The information derived from early settlers in the country of that handed down in Indian traditions must be obtained, and even after every effort has been made to learn the truth the deductions may be very far from correct. A record of thirty years' measurements by the U. S. G. S. is a good one as far as it goes, but is not enough, and very many streams have not been measured at all. It is unfortunate that the appropriations made in the past by the Federal Congress for stream gagings have been so meagre that very little has been done, at least in the West, to determine the characteristics of the streams.

Where no stream measurements have been made, and when no precipitation records are available, then information gathered on the ground as to minimum height of water, rainfall, or snowfall, and a more or less intelligent guess, are all the means available to determine the nature of the development. Data to determine spillway capacity required are not so difficult to obtain, as the high water mark for many years previous can usually be found, but the development of a stream depends upon minimum and not maximum flow, and unfortunately low water marks do not exist.

It has been stated that the governing factor in determining the capacity of a hydro-electric development is the minimum discharge of the stream, but this should be qualified by a consideration of storage. Few, if any, streams are not suited for some storage control, the amount depending upon the ratio between the minimum and mean usual discharge in the minimum year. This again is influenced by the amount of standby steam generation capacity installed or contemplated. The load factor of the plant must also be taken into consideration to arrive at a decision as to whether storage shall be provided to care for seasonal, weekly or daily variations.

Storage regulation to furnish a constant daily discharge would be ideal but in most cases the cost of storage works to impound extreme flood waters would be prohibitive.

Uses of Forecast of Water Supply

Forecasts of probable water supply are necessary in the absence of authentic stream discharge records before the economical development of a stream can be undertaken. In a few cases fairly reliable precipitation records of the U. S. Weather Bureau are available and from them fairly accurate determination as to stream flow may be made. This subject was discussed in an extremely interesting contribution to the Hydraulic Power Committee's Report for 1920-21 by Charles G. Adsit under the caption "Determination of Rainfall and Stream Flow in Analyzing Water Power Projects."

Where water power developed from stored water is relayed by stream power it is advisable to have some knowledge of the amount of water which may be relied upon in order that good judgment may be exercised in the purchase of coal supplies for use when a deficiency of water exists.

In many cases stream flow is augmented by storage in impounding reservoirs and it is very advisable, in order that the regulation of the reservoir may be successfully accomplished, to have some more or less accurate knowledge of the amount of flood inflow that may be anticipated. This knowledge is particularly necessary when the storage limit is clearly defined and any flooding of the lands above that limit might lead to damage suits.

It is a quite generally accepted theory that the use of water for irrigation dominates its use for power, and in many cases streams are developed for power that are already encumbered with irrigation rights. It is then advisable that the power user should have a forecast of the probable flow during the irrigation season in order that enough water may be provided for the irrigation requirements and at the same time that none be wasted, as in many cases such waste would be equivalent to the depletion of the coal pile.

Forecast of Water Supply

Many methods have been used to forecast supply, daily, weekly and seasonally, and for the purpose of this report the subject may be advantageously divided into four sections as follows:

(1) Forecasting the average run-off of streams for which no discharge records have been kept from precipitation records.

(2) Seasonal forecasting of stream run-offs or impounding reservoir storage from precipitation records.

(3) Seasonal forecasting of stream run-offs or impounding reservoir storage from snow surveys.

(4) Forecasting precipitation from records of ocean temperatures.

Precipitation may be determined from records of the U. S. Weather Bureau, but it would be well before placing too much reliance on such records to inquire somewhat as to the reliability of the observers at cooperative stations; the records from some observers would be very misleading, and where doubt exists as to such reliability a company would secure a greater measure of success by installing a gaging station under its own control. It is unfortunate for all parties concerned that the Weather Bureau regulations are very restrictive in regard to the establishment of cooperative stations.

The use of winter rise on open lakes that never freeze to determine the seasonal precipitations has been attempted but with very indifferent success. The fact that water is an inelastic standard and that all the snow that falls on the lake is retained in the form of water is an advantage, but the complications that enter into the problem are many and troublesome. Some of these factors are the winter run-off from the water-shed into the lake, the evaporation from the lake, and the winter discharge from the lake.

SECTION 1

Reference has already been made to the contribution of this subject by Mr. Adsit, and while the method developed by him is susceptible of application to other water-sheds, your chairman has been unable to ascertain that this has been done.

Dr. J. E. Church, Jr., of Reno, Nevada, reports that he has arrived successfully at a determination of the quantitative annual run-off of the East Walker River in Nevada in terms of the known run-off in the neighboring West Walker and seasonal percentage of snow cover. He has found it possible after one season's stream flow measurements on a stream to create a normal by expanding or diminishing said run-off according to the departure from normal in the neighboring stream for the same season; that is, in case a long record for this latter river has been kept.

SECTION 2

In January, 1921, a very interesting report was prepared by H. L. Stoner of the Utah Power & Light Company on Prediction of Bear River run-off above Bear Lake storage control from Precipitation Data. In this report Mr. Stoner describes the generating system of the Utah Power & Light Company which consisted at that time of thirty hydro-stations and one reserve steam station, four of the most important hydro-stations being located on Bear River below the Bear Lake storage reservoir. A particularly interesting feature of this storage system lies in the fact that, due to the capacity of the lake, it is possible not only to regulate the yearly run-off, but to carry over to the following year any surplus that may be left after the yearly requirements have been taken care of.

The main supply for the Bear Lake storage is furnished by the Bear River, the streams of lesser importance discharging only about enough to offset the yearly loss from evaporation.

The situation was such as to require some foreknowledge of the probable run-off which could be used for storage, and in that connection Mr. Stoner stated as follows:

> On account of the size of the drainage area and the inaccessibility of the greater part of it, the ordinary methods used in making snow surveys to determine probable run-off cannot be attempted. With such the case, several years ago studies were begun to determine if the data obtained by the existing voluntary cooperative stations of the U. S. Weather Bureau could not be used in forecasting probable run-off, the belief being that, though these stations were in the valleys, hence known not to have as great a snowfall as occurs in the mountains, would still act as "indicators" of the precipitation which occurs over the whole area. This, it has been found, is quite the situation: apparently the winter storms of consequence in the upper Bear River Drainage extend over large areas and fall upon the same elevations and similar slopes with considerable uniformity. Most of the storms of consequence come from the northwest or from north of west.

Mr. Stoner has developed curves showing the relation of precipitation for four periods to run-off for periods March—July. The precipitation periods are from November—January—February—March and April, and states:

> The purpose of the four arrangements is to furnish at as advanced a date as possible as much of a prediction regarding probable run-offs as can be ventured. From the relations shown in the comparisons, it is possible as the precipitation records are received from month to month to venture the following predictions with the quite positive assurance that the actual run-off performance will bear them out: At the end of January a prediction as to whether the run-off for the coming flood period will be low, average or high as compared to other years; at the end of February a verification or modification of the end of January prediction; at the end of March an approximation of the quantity of run-off in day-second-feet that will occur may be ventured; at the end of April a quantity estimate can be given which will no doubt be closely approached by the actual run-off. This last estimate can be given when the storage period is less than half elapsed. It should be noted that the comparisons provide quantity forecasts only. Rate and time of occurrence forecasts are not attempted.

In addition Mr. Stoner has developed curves showing the relation of the run-off March—July to that of the period following, August—February and states:

> From the relation found to exist between the run-off periods as expressed by the curves, it would seem apparent that the variation of run-off of one seasonal year with another is practically dependent upon the amount of precipitation during the previous winter period, considered as November-April, and that the precipitation which occurs outside that winter period, unless abnormal, has relatively little effect in varying the run-off. In other words, it would seem that the winter snows afford the main supply from which the entire year's run-off occurs.
>
> The purpose as stated above for attempting to show the relation expressed by the curves on the diagram is to permit estimating probable water supply as far in advance as possible. During the period August-February the flow of Bear River is relatively steady and not subject to

great range in stage. Hydrographs show much less variation in form during this non-flood period than occur during the flood period. Such being the case, it should be possible to estimate fairly accurately in advance the monthly run-off during the non-flood period by using the form shown by hydrographs of previous years, being guided as to total quantity by the relation shown in the diagram.

In the Great Lakes Division some of the companies maintain rain-gaging stations and stream-gaging stations in cooperation with the U. S. Weather Bureau and the U. S. Geological Survey. This work, however, is more along the line of obtaining the yield of different streams, knowing the amount of rainfall, rather than attempting to forecast the water supply.

C. D. Spencer, Chief Engineer of the Vermont Hydro-Electric Corporation, states:

We forecast our water supply at all our stations daily, monthly and yearly. Our daily water supply is forecast by daily precipitation records at different points on our system and by run-off records of past years.

Our water supply is forecast monthly by a study of the past records of both precipitation and run-off and the prevailing condition of stream flow and storage.

Our water supply is forecast yearly by a study of past records of precipitation and run-off and the condition of our storage. We have some monthly precipitation records which have been kept for twenty years and have daily storage records for about eight years. We find that we can forecast our water supply very closely. Of course no one can tell how much precipitation or run-off we are actually going to have each day, month or year, but we find it very essential to do this forecasting to know how to handle our storage.

Charles A. Mixer, Chief Engineer of the Rumford Falls Power Company, states:

The water supply of the Androscoggin River has been forecast for years. The practice is simply adapting to this case the methods anyone would use for a continuous inventory of any kind of material, like fuel, line hardware, etc., viz.: start with an inventory, know the probable demands and supplies from month to month, record all deliveries and receipts and periodically check by another inventory.

J. B. Mahoney, of the New England Power Company, states:

The New England Power Company System, in collaboration with representatives from the United States Geological Survey, has established accurate rating stations throughout the upper portion of the Connecticut River, where are also kept records of rainfall, snow, temperatures, etc. These observers record water heights at their respective stations twice daily as well as the amount of precipitation at 8 a. m., which records are mailed each day to the dispatcher's office. In case of precipitation of more than 1/10 of an inch the reading is immediately telegraphed to the dispatchers together with complete information covering the duration of the storm and general weather conditions at the time. During freshets or unusual weather conditions readings are obtained by telephone, and as a complete record is maintained of such precipitation reports a monthly average rainfall is computed over the drainage area covered by the reports. The gage readings are plotted twice daily on cross-section paper provided for this purpose, and constitute a continuous curve on the action of the river at any particular gaging station for any day in the year. From this curve the effect of atmospheric changes on the flow of the river can be determined at any given place. On this sheet the actual available kw-hr. flow at Vernon is also shown, so that a comparison between gage readings and kw-hr. flow at the Vernon development is also available. Similar methods are also employed in recording atmospheric conditions, flow and precipitation data on the Deerfield River; except that hourly readings of the gates in the forebays of the different stations are used in place of gage heights and precipitation records are kept by our own station operators, inasmuch as this river is so intensively developed our own records are available over the entire watershed.

In addition to the above records there are also kept in graphic form the following data: flashboard and forebay elevations; temperature and ice conditions during winter months; 24-hour output in terms of kw-hr., 24-hour flow of river at each location in terms of kw-hr., and also second feet; and draft on storage and available flow of plants in terms of kw.-hr. for various weekly load factors. These records are used very extensively in estimating river flows and predicting from day to day the amount of load each hydro-electric plant can carry, and to determine the minimum amount necessary to be carried by steam auxiliaries. In preparing this schedule many factors are considered, such as weather predictions and conditions, ground conditions, plant conditions, available water storage, steam auxiliaries, estimated load, economy of different steam auxiliaries, plant effective head and, in the winter months, ice and fluctuating temperatures; all of which play important parts in the efficient production of power. Should weather predictions indicate rain in the near future plans are made accordingly, and in such manner that in the event of no rain, normal operations may be resumed with the least possible loss in economy. In case of actual precipitation and an imminent increase in river flow, the problem then becomes one of estimating and handling the flow with as little loss and inconvenience to such steam auxiliaries as we have had prepared to pick up load as is possible.

This same holds true in case of decreased river flows, schedules being arranged in accordance with estimated river decreases from information obtained from gage readers, as mentioned above. During periods of indefinite weather a daily check is made with the Weather Bureau in addition to the regular reports received from observers by mail. The condition of the ground and amount of ground water determines, to a great extent, the available run-off during periods of no precipitation as well as the available percentage of actual rainfall measured in c. f. s., during periods of precipitation. It is, therefore, necessary, in order to estimate the probable flow of the rivers with any degree of accuracy under all conditions, to keep a daily check on the cubic foot flow at the various stations, which is converted into kw-hr. During the fall and winter months great changes are effected in the river flows by variations in temperature; in the spring by thaws and rains, and in the summer by sudden, short, heavy showers, as well as prolonged hot weather. Storage water being comparable in value to a coal pile, it is essential that it should be used with the utmost efficiency.

Plans are made in the early summer whereby storage reservoirs will be drawn sufficiently by the following spring to entrain all spring rains and run-off from their particular watersheds. Estimates are then made whereby the available storage is distributed advantageously over the minimum flow months in accordance with the estimated yearly load requirements and river flows. Economy in operation in hydro-electric plants is as essential as in steam-electric installations, and the same study and care is given to the removal from service of unnecessary machine, transformer and condenser equipment as would be given to the latter. In addition, the utmost precaution is followed in avoiding the drawing of ponds in such a manner as to incur unnecessary head losses, and the use of such wheels as show bad leakage conditions either for load or condensing equipment for power factor correction.

George S. Williams, General Superintendent, Central Maine Power Company, states:

Most of our storage lakes and ponds have been handled by the same personnel for years and through use a reasonably close knowledge has been obtained.

The capacity of each pond is known in kw-hr. output. The draw down and water level the following morning is anticipated. This is especially true of one of our most important plants.

The water in a portion of the storage areas is held until the latter part of the summer, until water used for log drawing on other streams is cut off. The amount let out is dependent on judgment of forthcoming weather conditions.

Gage readings and gate openings are taken daily at several important lakes.

From observations of last year on one river we found that waste of water was in a large measure due to lack of knowledge of extent of each rain, and immediate operation of sluice gates. If, for instance, there was desired a flow of 2800 sec. ft. and there was being sluiced 1500 sec. ft., a portion of gates or all should be closed, depending on the amount of rain to prevent water running to waste. Oftentimes for several days the lower drainage areas would supply the required flow for several days after each rain.

All use of storage is dependent on rainfall which is impossible to accurately anticipate.

We have attempted to make a study and curves for regulation, but they have not been put into practice. It seems evident that certain control and information may be obtained by plotting the flow and storage volume for a median year, a wet year and a dry year, then plot the actual storage from week to week and see how it compares. If it should pass under the median year it would be reasonable to cut down the amount of water sluiced. If it still continues to approach the dry year it would still further have to be cut down, or vice versa.

Several other companies in the New England Division report that they are maintaining stream-gaging stations and are interested in forecasting water supply but have not yet attempted the doing of any such work.

SECTION 3

While snow surveying for the purpose of forecasting water supply has not yet been generally practiced to any great extent in this country, it has been practiced for over ten years in Utah, Nevada and California, and in Switzerland.

Two principal methods have been developed. The percentage method developed by Dr. J. E. Church, Jr., of the University of Nevada at Reno, is followed in the Lake Tahoe basin, in the Sierras in connection with the Nevada-California Cooperative snow surveys and by the Washington Water Power Company in the Cœur d'Alene watershed, and the method of areas which has been followed to some extent in Utah.

The percentage method consists essentially of a determination of the percentage variation from the normal of the water content of the snow blanket and the assumption that the stream run-off from these snow fields will have the same variation from normal. It is advisable to establish a number of courses in any watershed depending upon the variation in the character of the mountains, the variation in the snowfall in different portions of the watershed, and the difference in elevation. On each snow course at distances of approximately 50 ft. apart about forty samples of snow are taken by means of a Mount Rose Snow sampler, and these samples are weighed on spring scales calibrated in inches of water instead of in pounds. An average of all the readings from all the snow courses is taken and this is assumed to give the average depth of water over the entire snow field. In forecasting the run-off consideration has to be given to the meteorological conditions prevailing during the melting period. This method has given very satisfactory results over a considerable period of years.

The method of areas was developed about ten years ago by Salt Lake City in cooperation with the U. S. Weather Bureau and has been followed in the snow surveys in the Big Cottonwood Canyon, Utah. This method consists in dividing the watershed into areas and, by sampling the snow in each area, determining the number of acre feet of water in the whole snow blanket. From this amount have to be deducted the estimated losses due to evaporation and the priming of the soil. The results obtained by this method have not been satisfactory.

A third method is that of accumulated snowfall from which only approximate results may be expected. One of the factors which detracts from the accuracy of this method is the lack of knowledge as to the density of the snow fom storm to storm and from season to season. It has the advantage of very low cost.

A fourth method approaching in accuracy and being somewhat cheaper than snow surveys, is the depth of snow on the ground. It does not take into account the density at the time of measurement, which will vary from season to season with the winter weather, particularly the temperature and the wind, and any new snow must be allowed to settle to approximately the same density as the snow beneath before readings are taken. It has been found that with the proper precautions taken the variation in density from season to season will seldom exceed 10 per cent. To obtain reliable results snow courses practically the same as those used for snow surveys must be established, and as a result the only saving is the moderate cost of the snow samplers.

The most extensive work of snow surveying which has been carried on in this country is that of the Nevada-California Cooperative Snow Survey, which was inaugurated about twelve years ago by Dr. J. E. Church, Jr., of the Mount Rose Observatory in Nevada, and which has been carried on continuously under his direction.

The territory covered lies in the Sierra Nevada Mountains, embracing the Tahoe, Carson and Walker basins on the eastern slope and the Yuba, American, Mokelumne, Stanislaus and Tuolumne basins in the western slope, and in the Charleston Range and Ruby Mountains in Northern Nevada, embracing the Humboldt and Little Humboldt basins.

About October of 1922 arrangements were made for an extension of the cooperative snow surveys to the southward to include the main streams on

both slopes of the southern Sierras. Cooperating in this work will be the City of Los Angeles, the King's River Water Storage District, the Southern California Edison Company, and the Southern Sierras Power Company.

For three seasons the Washington Water Power Company has made snow surveys, under the direction of your Chairman, in the Bitter Root Mountains embracing the Coeur d'Alene, St. Joe and St. Maries basins. As it requires several seasons' records to establish reliable normals, it is impossible to say at this time what the results will be, all that can be said is that they promise well.

The Bitter Root Irrigation Project is planning to inaugurate snow surveys and will probably adopt the Mount Rose sampler for the purpose. The territory to be covered lies in the Bitter Root Mountains to the southwest and not very far from the territory covered by the Washington Water Power Company survey.

The U. S. Reclamation Service at American Falls, Idaho, has ordered a Mount Rose Sampler for use in snow surveys to be conducted in the Yellowstone National Park.

It is the understanding of your Chairman that about the year 1917 snow surveys were inaugurated by the Canadian Meteorological Service to enable a forecast to be made of the discharge of the Bow River in Alberta, Canada. Whether or not the work has been kept up, he is unable to state, as inquiries made of the Canadian Division have failed to bring any results.

SECTION 4

Where the source of water supply is not from mountain snows but from winter and spring rains, there is a possible method of predicting water supply other than that described by Mr. Adsit and referred to herein. For some time past observations of oceanic and atmospheric relations have been conducted by the Scripps Institution of Biological Research of the University of California, and these lead to the belief that there is a possibility of developing a method of forecasting season's rainfall. The relation is one of ocean temperatures to rainfall, the ocean temperatures being observed at the Scripps Institution pier at La Jolla and the precipitation at San Diego, Escondido, Bonita, Los Angeles, Tustin, and Corona. A simple empirical formula has been developed, and by means of this formula predicted rainfall has been computed.

An article by George F. McEwen, who has been carrying on the investigations, was published in the *Bulletin* of the American Meteorological Society for October, 1922, and contains the tabulation presented below:

While the results as shown in the table show only a fair agreement between computed and actual departure from the mean precipitations, they are sufficiently close to be at least very interesting, and Mr. McEwen in his article states: "The results obtained to date may be regarded as the first crude approximation to the lengthy and detailed investigation that should be made."

Conclusions

No one can take up the study of the forecasting of water supply and fail to reach the conclusion that literature on the subject is very scarce; that many hydro-operators are failing to take advantage of the knowledge that has been accumulated on the subject or are ignorant of it; that where water in storage is relied upon for generating purposes a knowledge in advance of probable inflow to reservoirs would be of great advantage; that in very many cases a fairly reliable forecast of water supply can be made by one method or another; and that the subject is deserving of further investigation and study.

For assistance rendered him in the preparation of this report your Chairman desires to express his acknowledgment and thanks to J. B. Mahoney, New England Division, to Stanley B. Wiggins, Great Lakes Division, to H. L. Doolittle and R. J. C. Wood, Pacific Coast Electrical Association, and especially to Dr. J. C. Church, Jr., Mount Rose Observatory, Reno, Nevada, who, though not a member of the Association, is so deeply interested in the subject that he spent considerable time and effort in furnishing desired information.

Year	Temp. Departure from 6-year mean	Bonita San Diego Escondido Rainfall for the season Departure from 6-year mean	Tustin Corona Los Angeles Rainfall for the season—Departure from 6-year mean	Mean of all six Stations Rainfall for the season—Departure from 6-year mean	Computed departure from the 6-year seasonal mean
1916-17	66.4 —1.2	12.8 +0.2	13.0 —0.2	12.9 0.0	—2.9
1917-18	68.8 +1.2	10.0 —2.6	11.7 —1.5	10.9 —2.0	—2.9
1918-19	69.3 +1.7	9.6 —3.0	8.1 —5.1	8.9 —4.0	—4.1
1919-20	66.7 —0.9	11.7 —0.9	12.6 —0.6	12.2 —0.7	—2.2
1920-21	67.8 +0.2	9.2 —3.4	12.5 —0.7	10.8 —2.1	—0.5
1921-22	66.4 —1.2	22.3 +9.7	21.0 +7.8	21.7 +8.8	+2.9
Means	67.6	12.6	13.2	12.9	
1922-23	67.8	?	?	?	—0.5

EFFECT OF ICE ON FLOW OF MISSISSIPPI RIVER AT KEOKUK, IOWA

An interesting effect of ice on stream flow is that which occurs annually on the Mississippi when that stream is changing from its open-water regime to its regime under a solid ice cover. The experience at Keokuk is consistent from year to year because the natural flow of the river is not affected to any great extent by the operation of power plants above. Besides this there is an opportunity to measure accurately through the wheels the flow from day to day.

Another thing which makes the situation at Keokuk rather unique is the size of the stream, its low velocity, the absence of riffles or controls in the channel, and the long distance from Keokuk to the main sources of sustained flow, which come mainly from the Wisconsin and Minnesota lakes. Since the main source of supply is so far from Keokuk it takes a rather long time a matter of three weeks, for the complete change from open water to closed channel flow to take place.

This very definite and measurable ice effect at Keokuk is described herein with the idea that it may throw light on the general problem of flow under ice cover in other locations. There is considerable literature concerning methods of determining the flow of an ice-covered stream by flow measurements and observations of gage heights, ice thickness, etc., but little material, that the writer is familiar with, to explain the causes of the changes of flow. It is obviously impossible to obtain, by current meter, measurements of flow while there is running ice in a stream or when the ice is unsafe. For this reason largely, we have had little knowledge of the changes in flow that take place in the transition period.

The first running ice appears in the river at the head of the lake about December 8th in the average winter and a solid ice cover on the river north of Keokuk completely forms in about six days. The river north of Keokuk then remains completely closed until about March 1st when the beginning of the final break-up sets in. Altogether, the period of ice effect persists for about 82 days. Out of this the first 20 days represent what is termed at Keokuk the "initial freeze" period.

The operating experience of the last ten years has yielded excellent records of flow and has given ample proof of the following conclusions relating to the Mississippi.

1. That the flow during any winter period will depend mainly on the flow at the time of the first freeze-up.

2. That the initial freeze-up will cut the flow down to 40 per cent of the original flow in six days; it will keep it down close to that figure for five days and then allow it to recover gradually in nine days to a flow between 75 per cent and 80 per cent of the original flow, the average period from freeze-up to full recovery being about twenty days.

The reason for the sudden drop-off in flow is that the blanket of ice holds in check the velocity of the water throughout the drainage. The wetted cross section later increases to compensate for the reduced velocity. This results in a considerable amount of storage in the form of water, entirely aside from the ice storage, which water is released in a few days when the ice blanket is lifted off the streams in the spring. This is the underlying cause of the floods which invariably accompany the break-up.

The theory developed at Keokuk is that under a solid ice cover in a stream without a well defined control the flow will be only about one-half the open water flow for the same water surface gage height. The flow in an open river begins to decrease as soon as light running ice appears and continues to decrease until a solid ice cover has formed. As stated above, on the Mississippi it ordinarily takes 6 days of weather with temperatures below freezing to develop a solid ice cover.

Immediately after the solid ice cover has formed the flow remains constant at somewhat less than one-half the open water flow until the source supplying the stream has had time to fill up the additional storage area which is required under the ice cover to accommodate the normal winter flow at the decreased velocity. The volume of water which is held back over the "initial freeze" goes to increase the cross sectional area of the stream throughout its length. It is this same volume of water which is released suddenly when the ice breaks up in the spring.

The underlying cause of all this is, of course, the fact that an ice cover practically doubles the "wetted perimeter" of a given cross section and doubles the area of the surfaces producing friction. A general consideration would lead one to expect the velocity of the stream to be halved. While a theoretical consideration of the Chezy and Kutter formula does not show that the velocity is halved by doubling the wetted perimeter, actual experience shows that this occurs. The conditions of flow under an ice cover are so different from open channel flow that it would hardly be expected that the constants of one would apply to the other. Flow under an ice cover is more comparable with flow in pipes or closed conduits running full.

In the U. S. Geological Survey stream gaging work in southeastern Iowa it has been found that the flow under an ice cover in a stream without well defined controls averages very close to one half the open water flow for the same gage height. It is true that in many streams the presence of open riffles, power dams, non-uniformity of ice cover, ice jams, etc., may entirely obscure the normal effect above mentioned. In the streams of the middle west, however, and especially the Mississippi, the effect is very regular.

As proof of this, it might be said that during the past winter the flow at Keokuk was predicted for 7 weeks in advance with an accuracy of 1 per cent

for the total flow in the period. The maximum error on any one day was less than 10 per cent, this error being 2,300 second-feet on January 9th when flow was 24,300 second-feet. The prediction was borne out from day to day in spite of big variations in mean temperature. The prediction was made on December 8th for the entire period up to the present date, January 25th. In this period the maximum temperature varied from 16 deg. to 63 deg. and the minimum temperature from 2 deg. to 40 deg. At no time, however, did the ice covers on the streams threaten to move out.

This experience is cited merely to show that flow over the "initial freeze" period obeys laws as definite as those governing open channel flow. It is certain that there is no time in the whole year when the flow at Keokuk may be predicted far in advance with more accuracy than during the winter period.

The idea of water being held back by a series of ice jams at the beginning of the freeze-up is a common and convenient one but it is not the main cause.

In further confirmation of the effect of a solid ice cover on velocity it has been observed repeatedly in the 40-mile lake above the dam that back water slopes under a reasonably smooth ice cover are practically twice the slope under open water conditions where the flow is the same. Since there is no appreciable change of cross section in the lake it is apparently necessary to have twice the slope to produce the same velocity. This is merely the converse of the statement above that in a normal stream where the slope cannot change the velocity is halved by the presence of an ice cover.

A matter of interest in connection with the prediction of winter flow at Keokuk is the way in which the water temperature and up-river stations are watched to anticipate the formation of ice and the consequent drop in flow.

It has been found that the flow begins its drop on the day following the appearance of floating ice in the main river at a station about 170 miles above Keokuk; that the drop begins in earnest when floating ice is reported from Keithsburg, 65 miles above the dam; that floating ice appears in the Iowa streams on the day preceding the appearance of floating ice in the main river at Keithsburg.

From a study of water temperatures it has been observed that as soon as the river water temperature has fallen to 35.0 deg. F. with temperatures well below freezing, floating ice will appear. By watching water temperatures it is possible to know when either surface or frazile ice *cannot* form and to know when conditions are favorable for their formation.

The water thermometer used is of the direct reading mercury type but graduated to tenths of degrees Fahrenheit and capable of being read to hundredths of degrees by estimation. It is enclosed in a water thermometer case of the standard U. S. Weather Bureau type. Regular readings are taken every 3 hours and special readings every half hour as necessary.

Several operating companies have in connection with this article very kindly recorded their experience. Their experiences coincide with conditions at Keokuk in a general way, but irregularities of low from power plants and other factors tend to obscure the normal drop and recovery in flow as observed at Keokuk.

The Northern States Power Company reports as follows for its Coon Rapids plant on the Mississippi.

Following a very cold period with high wind, where the river has had a chance to cool to the freezing point throughout its area north of Coon Rapids, the freeze-up may come in remarkably quick time and cause a drop of 60 per cent to 70 per cent of normal. Our recovery, however, following the freeze-up does not approach that indicated at Keokuk. It is influenced more by the condition of the swamps and the underwater supply, which is, of course, dependent on the amount of precipitation preceding the freeze-up.

H. H. Cochrane of the Montana Power Company, with plants on the Madison and Missouri Rivers, reports as follows:

Your description of the so-called initial freeze applies very well to our conditions, except that on account of the fact that our rivers are shallower and swifter flowing, the ice does not freeze across solid so readily. We have low enough temperatures, however, so that plenty of ice forms. The result is that during extreme cold spells the flow is reduced not so much on account of the added friction of the ice cap as due to the fact that the floating ice jams and gorgers at certain places, backing the water up and flooding the surrounding country. This overflow water then freezes, and never does flow down the stream again until it melts.

J. B. Mahoney of the New England Power Company reports his experience as follows:

It would be extremely difficult to give a definite value to the drop flow of our rivers caused by temperatures below freezing, as this quantity is not only regulated by storage water on both the Connecticut and Deerfield Rivers, on which our plants are located, but also by a number of power and industrial installations located on each of the two rivers in question. There is, of course, a very distinct drop in flow when temperatures drop below the freezing point, and the drop is dependent on the flow at the time of the freeze as well as the length of the cold spell and extent of the temperature drop. We estimate the drop and flow by comparison of conditions existing at that time, with the previous action of the rivers under similar conditions and, if it is found that the drop is to be of sufficient magnitude or duration to warrant it, storage water is used to build up the flow, or the output of the hydro-electric stations is curtailed and made up by steam auxiliaries.

Temperature drop seems to have its greatest effect on the flow of our rivers below 20 degrees, at which time we not only observe the greatest loss in quantities, but also a very distinct change in velocity. We also note a heavy snowstorm affects the flow of the rivers to some extent. It has been our experience that after the first drop in flow, due to temperatures falling below the freezing point, the flow remains practically constant until the temperature falls below 20 degrees, at which time the greatest drop in flow is experienced, the extent depending, of course, on the actual drop in tmperature.

The experience of the Pennsylvania Water & Power Company at its Holtwood plant given below would seem to be identical with that at Keokuk in a percentage way. The time interval from begin-

ning of the freeze-up to recovery is much shorter at Holtwood than at Keokuk. Their flow in December, 1917, was roughly one-half that at Keokuk. Before the freeze-up, it averaged one-half that at Keokuk at the bottom of the drop and one-half the flow at Keokuk after the recovery. The duration of the so-called "initial freeze" at Holtwood was 10 days as compared with 20 days at Keokuk. It is to be assumed that their plant is nearer the main sources supplying their stream.

H. W. Lowy of that company gives their experience in the following words:

At the beginning of the freezing season of the year we are prepared for a sudden drop in the river flow after the initial freeze-up, which generally extends over a week or more. We have observed at such times that this is first accompanied by a rise in gauge height reported at Harrisburg, generally of one or two tenths, though the tendency of the river might be to drop. We then expect a sudden drop of 30 to 60 per cent of the initial flow. After that the river flow to remain low or in case of very low temperatures dropping in excess of the previous sudden drop for several days and then to recover within 20 per cent of the flow prevailing at the time of the initial freeze-up.

It generally takes about five days of temperatures below freezing to cover the river with ice extending to the dam at Holtwood, with the exception of some open riffles near Safe Harbor, which sometimes remain open during the entire winter. The flow of the river begins to fall as soon as ice appears on the river, mostly on account of the reduction of cross section. This reduction of cross section is the cause of a receipt of generally higher gauge height readings from various up-river stations.

Altogether it is hoped that the above record of experiences and suggestions as to the underlying causes will prove enlightening and help in the estimation of stream flow during winter periods when the critical flow of the entire year may occur. Many prime water power plants have been projected on the basis of open water flow records only to find a serious water shortage at the beginning of the winter period.

NATIONAL HYDRAULIC LABORATORY

As the interest of the Association in the proposed national hydraulic laboratory now under discussion at Washington has been questioned, it was thought worthwhile for this Committee to investigate and prepare a report on a subject which would illustrate one of the uses of such a laboratory and would at the same time be of interest to the engineers of most of the member companies. The report prepared by R. L. Thomas on the "Use of Ogee Dams as Measuring Weirs" shows the great diversity of methods used by the various companies in computing spillway capacity. Uniform practice is a matter which can be worked out to the entire satisfaction of the industry only through the medium of some agency such as a national hydraulic laboratory.

The Use of Ogee Dams as Measuring Weirs

The tendency in hydro-electric plant operation is toward a strict accounting of water supply and the use made thereof. Ogee spillways often are the best, and in some cases are practically the only means of measuring the flow in excess of the turbine discharge. At low head plants in particular it is a convenient and common practice to compute the total flow by adding to the power house discharge the estimated flow over the dam as given by a rating curve or table, with the proper correction for change in storage, leakage, etc. Many companies put their main reliance on a gaging station at a well established river section above or below the pond—a practice which, with our present knowledge of overflow rating, is very desirable. In many cases, however, on account of topography, character of the channel, amount of water involved, location, etc., it is not feasible to establish a gaging station. Furthermore even where there is an independent rating section it may be almost impossible to extend the curve to flood discharges with any degree of satisfaction, and use must be made of the spillways.

When the operating engineer attempts to establish a rating curve for a spillway he finds a surprising lack of information. Both the theory and experimental data are inadequate, and a regrettable feature of the situation is that little progress seems to have been made in the last ten or fifteen years.

Inquiry among the companies represented on this committee showed that there is a wide discrepancy in the methods of computing overflow, even for dams of a similar type. Investigation and research on a technical subject of this nature are not functions of this committee, but it seems worth while to point out the problems, with the earnest hope that someone will undertake to carry out much needed experimental and practical research.

Practically all formulas for computing flow over dams are modifications of the well-known Francis

$$Q = C\,L\,H^{3/2}$$

formula in which, for a sharp-crested weir, C is 3.33. To adapt the formula to broad crested weirs the coefficient may be changed, the head may be modified, or the exponent may be changed. A theoretical method which is sometimes used in designing a dam to withstand the maximum pressure and to give the ogee curve the proper form is to assume that the weir crest is at the top of the upstream face, or at the beginning of the crest curve, instead of at the crown of the curve. However, this method is applicable only at maximum discharge, and when the dam was designed with this in mind.

The best work on the subject still seems to be Water Supply Paper No. 200 of the United States Geological Survey by R. E. Horton (1907). This paper gives the results of several series of experiments and formulas for various dams. Mr. Horton deduces the following general formula for ogee dams:

$$Q = (3.62 - 0.16\ (S-1)\ )\ L\ H^{1/20}\ H^{3/2}$$

$$\text{or } Q = (3.78 - 0.16S)\ L\ H^{1.55} \qquad \text{(p. 131)}$$

where S is the slope of the crest curve rise, being horizontal run over vertical rise.

The formula was deduced from a comparatively small number of experiments on a few models of dams of a particular type. The author himself calls the equation "a convenient approximate formula, applicable for weirs with 2 or 3 feet crest radius and upstream slopes 3 to 4.5 feet broad." The application of the formula to broader dams for heads up to 18 or 20 feet, as has been done, is probably unwarranted.

In *Engineering News* for Sept. 29, 1910, p. 321, W. F. Martin gave the following formulas for three dams, deduced from extensive stream gagings for the U.S.G.S.:

Merced Dam	$Q = 2.491\ L\ H^{1.77}$
Austin Dam	$Q = 3.196\ L\ H^{1.59}$
Yakima Dam	$Q = 3.110\ L\ H^{1.65}$

(Note: LaGrange Dam is here omitted, as it is curved in plan.)

An interesting German method, deduced by Prof. Rehbook, makes the coefficient a function of the ratio of head to dam height ("Der Wasserbau," Leipzig, 1912, p. 51).

A discussion of these and other theories is beyond the scope of this report, but the following abstract of replies to an inquiry sent to certain member companies is of interest, particularly as showing the wide divergence of method:

COMPANY A:

This company has an ogee dam surmounted by gates 30 ft. long. The piers between the gates, 6 ft. in width, have pointed concrete cutwaters or noses.

The company uses a rating curve, made prior to the erection of the dam, for a gaging station located just below the dam. The dam was calibrated as a weir by taking simultaneous readings of the lower gage and the forebay gage and deducting the turbine discharge from the total discharge, as shown by the rating curve, thus obtaining the discharge over the spillway. These readings were taken with various numbers of spillway gates open.

From this discharge the value of C in the Francis formula was found to be 4.1 to 4.3 for heads of 13 to 14 ft.

The value of 4.2 is now used for C for all heads from 10 to 15 ft. without correction for end contractions, velocity of approach or number of gates open.

COMPANY B:

This company has an ogee dam surmounted by gates giving a clear spillway width between piers of 30 ft. The piers, 6 ft. in width, have cylindrical upstream noses.

The dam is used in making routine computations of overflow. The Francis formula is used with the following values of C:

For	1.0 ft. depth of crest, C =	3.15
	2.0 ft.	3.18
	3.0 ft.	3.19
	5.0 ft.	3.26
	8.0 ft.	3.31
	11.0 ft.	3.33

No correction is made for velocity of approach or end contractions.

Stream gaging which has been done by this company since the establishing of these coefficients indicates that these values are too low by 5 to 10 per cent.

COMPANY C:

Ogee dams are used as measuring weirs during flood water periods. The Francis formula is used. C varies from 2.5 at the first overflow to 3.5 at 2 ft. and higher heads.

No correction is made for velocity of approach.

When flashboards are placed on the dam the Francis formula is used with $C = 3.33$.

COMPANY D:

This company has one ogee spillway which passes the spring flood each year and is used to measure the overflow. The spillway is divided into bays by vertical I-beams spaced 6 ft. on centers for holding flashboards which are placed as soon as the first flood has sufficiently subsided. As the only water which is measured in this manner is waste water, accuracy of measurement is not needed.

The Francis formula is used with $C = 3.33$, both before and after flashboards are placed. The length of spillway is corrected for full end contractions at each bay, thus:

$$Q = 3.33\ (L - 0.2\ H)\ H^{3/2}$$

No correction is made for velocity of approach.

COMPANY E:

This company has an ogee dam without piers or permanent gates. Flashboards are erected in low flow by inserting pipe pins in sockets in the dam crest. The dam is used as a measuring weir in all flow computations, no satisfactory nearby stream gaging station being available. The formula developed by R. E. Horton in Water Supply Paper No. 200 as previously given is used. As the slope of the crest curve rise is 2.0 the formula becomes:

$$Q = 3.46\ H^{1.55} \text{ (for 1 ft. length)}$$

In making up the rating curve for the dam the head was corrected for velocity of approach.

When flashboards are on the dam the Francis formula is used with $C = 3.33$.

COMPANY F:

This company has an Ambursen type dam with upstream face sloping at an angle of nearly 45 degrees followed by an ogee curve downstream. This is used for flow measurement only during extremely high floods when current meter measurements are impracticable. The Francis formula is used with $C = 3.77$.

No correction is made for velocity of approach.

COMPANY G:

This company has an ogee dam without piers or permanent gates. In low flow flashboards are placed on the dam by inserting 3½ in. pipe pins in the dam on 3 ft. centers.

An attempt is made to compute the amount of water passing over the dam at all seasons, but it is felt that the flow may be more accurately determined in high water from the government records at nearby gaging stations.

The Francis formula is used with the following values of the coefficient C:

Head	C
0.5	3.3
1.	3.46
1.5	3.56
2.	3.62
3.	3.70
4.	3.73
5.	3.77
6.	3.8
7.	3.84
8.	3.86
9.	3.88
10.	3.9
11.	3.9

The head is corrected for velocity of approach.

When the pins are in place without flashboards, correction is made for end contraction, but as the corrections are not considered complete only 50 per cent of the usual correction is applied, the formula used for correcting the length of spillway being as follows:

$$\text{Effective length} = \left(L - \frac{NH}{10}\right)$$

where L = Length between pins,
N = Number of end contractions and
H = Head on weir corrected for velocity of approach.

Practically all of these companies reported that the gage, from the reading of which the head is computed, is located far enough upstream to be clear of any effect from the drop-down curve at the dam. Practice in this respect cannot be uniform as the location of the gage in each case depends on local conditions.

Those companies which erect temporary flashboards supported by pins find it very difficult to compute the flow with any degree of satisfaction when pins are on the dam without the flashboards. The pins cause partial end contraction and also collect debris which interferes with the discharge.

The following tabulation presents a comparison of the rating given in the replies abstracted above at a head of 11 feet with no end contractions:

	C. F. S. Per Foot of Spillway Length	Equiv. Value of C in $Q = C L H^{3/2}$	Per cent of Rating for Sharp-Crested Weir
Company A	153.2	4.20	126.1
Company B	121.5	3.33	100.0
Company C	127.7	3.50	105.1
Company E	142.3	3.90	117.1
Company F	137.5	3.77	113.2
Company G	142.3	3.90	117.1
Sharp-crested weir (by Francis)	121.5	3.33	100.0

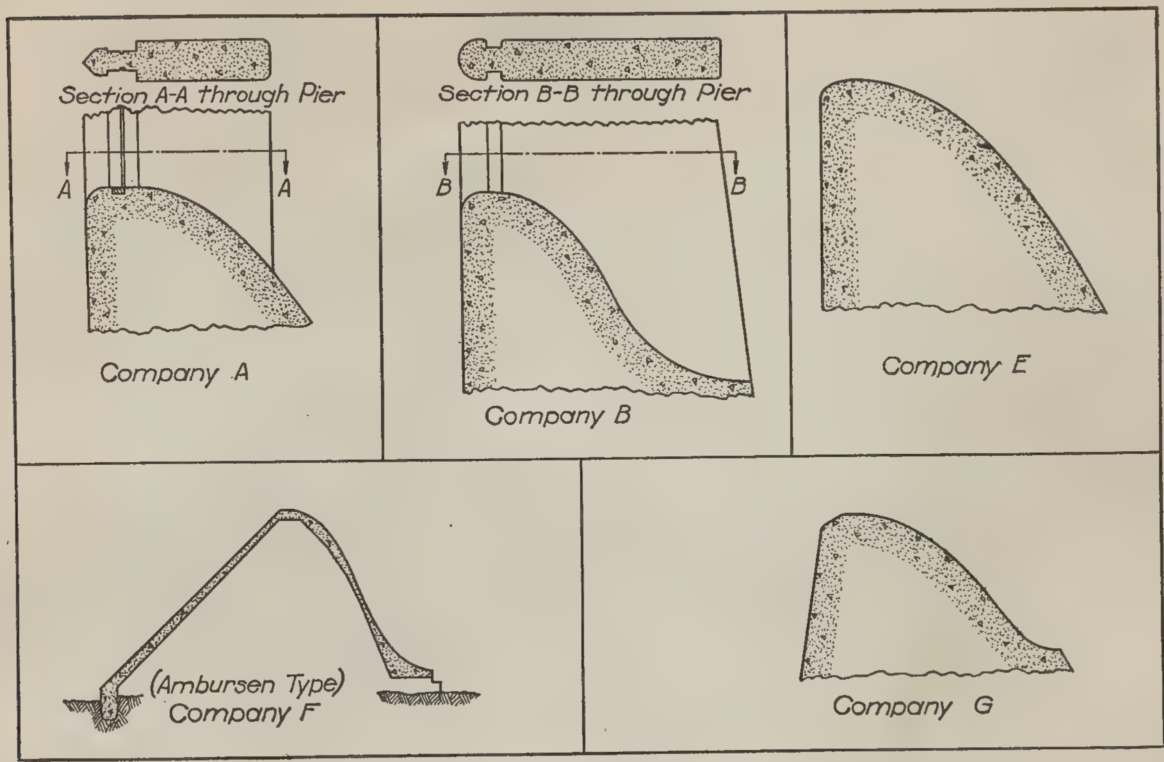


FIG. 42—Sketches Showing Crests of Dams Used in Comparison of Overflow.

The accompanying sketch (Fig. 42) shows the shape of the crests of five of the dams.

It will be noted that there is a difference of 26 per cent in the computed ratings and that the dam which is given the highest rating is apparently the only one which was calibrated by direct comparison with a previously rated gaging station (see abstract for Company A.) A part of this discrepancy is probably due to the shape of the cut-water and pier of Company A's dam. These are so formed that the net effect of the pier is probably not as great as an actual reduction of the spillway length by the thickness of the pier. Nevertheless the variation in method is so wide and actual data are so inadequate that this entire subject should be given exhaustive study. It is a problem which might best be handled by a national hydraulic laboratory, and illustrates the fact that such a laboratory could be of great practical benefit to the hydraulic art.

STATEMENTS OF MANUFACTURERS

Relative to Developments in the Hydraulic Field in 1922

Statement By William Cramp & Sons Ship Engine Building Company

In the field of hydraulic power development, the year 1922 has been perhaps chiefly notable for the carrying into actual construction on a large scale of some of the new ideas in turbine design which had been in the course of development during the preceding two or three years. Probably the two most notable installations under construction during 1922 are the following:

First: The three 70,000 horse power hydraulic turbines for the Niagara Falls Power Company's Station No. 3 Extension. These are the highest powered units which have ever been undertaken, and while they do not represent any radical departures in general type or arrangement, they will include numerous innovations in various elements of the turbines and auxiliary equipment. Two of these units are being furnished by the I. P. Morris Department of the William Cramp & Sons Ship & Engine Building Company and the third by the Allis-Chalmers Manufacturing Company. These will be, as far as we are aware, the most powerful prime mover units in the world.

Second: The 28,000 horsepower turbines for the Manitoba Power Company. These turbines are remarkable as being the first installation on a large scale of the new propeller type of turbine. Their design comprises radical departures in both general features and details. The first of these units has just been placed in commercial operation and its performance has been notably successful in every particular, fully justifying the faith of the designers and the engineers of the Power Company in taking this forward step on so large a scale. It is believed that the success of these units will mark a distinct step in hydraulic power engineering throughout the low and medium head field. These units were built by the Dominion Engineering Works, Limited, of Montreal, in accordance with I. P. Morris designs and engineering.

Another installation of note, the construction of which has been undertaken during the year, is the 35,000 horsepower turbine for the Oak Grove development of the Portland Railway, Light & Power Company, marking the adoption of a reaction turbine for an effective head of 849 feet. This unit is being designed and built in San Francisco by the Pelton Water Wheel Company. The principal features of the design are the result of a coordination of engineering between the Pelton and I. P. Morris organizations.

Among the installations put into operation during the year are included the first four 55,000 horsepower units at the Queenston plant of the Hydro-Electric Power Commission of Ontario, these being the most powerful units which have ever been placed in operation; the 41,500 horsepower turbine for the Shawinigan Water and Power Company; and the 40,000 horsepower turbines for the Mt. Shasta Power Corporation. The erection of the fifth unit at the Queenston station is nearing completion and it will be placed in operation at an early date. The first two Queenston turbines were built by the Wellman-Seaver-Morgan Company, the last three units by the I. P. Morris Department of the Cramp Company; the Shawinigan unit is an I. P. Morris design built by the Dominion Engeering Works, Ltd., and the Mt. Shasta turbines were furnished by the Allis-Chalmers Manufacturing Company. The Shawinigan unit stands second to the Queenston turbines in the list of the most powerful turbines in the world and in point of dimensions exceeds the Queenston turbines; it is believed that the Shawinigan unit contains the largest cast casing so far completed.

Among the developments carried out or started during the year, the following may be mentioned as exemplifying particular features of interest.

The construction of a fourth I. P. Morris turbine for the Long Lake Station of the Washington Water Power Company has recently been started. This unit will be of the same design and capacity as the previous units in this station, namely, 22,500 horsepower. The ordering of this unit illustrates the present tendency of the power companies to conserve water power more completely than ever before. This unit will be available as a spare and for carrying peak loads during most of the year and water will be available for its continuous operation during only a portion of the year. The value of increased station capacity both for conserving a greater portion of the annual run-off and for carrying temporary peak loads justifies under present conditions what would have been considered an over-development of water powers a few years ago, and the uncertain supply and high cost of fuel will probably justify similar expansions in other plants.

Another plant placed in operation during the year by the Washington Water Power Company is the new station recently put in operation in the City of Spokane. The station is interesting as illustrating the modern tendency toward the use of a small number of units, this particular installation containing only a single unit, an I. P. Morris turbine of 14,250 horsepower, operating under a head of 64 feet. This plant also contains an interesting feature in the provision of a large Johnson valve capable of bypassing water directly from the penstock to the river below in order to supply other stations downstream during times when the unit is shut down. This Johnson valve is of the free discharge type and is equipped with a provision for automatic opening when the turbine gates are closed below a given point.

Another single unit station put in operation during the year is that of the Power Construction Company at Searsburg, Vermont. This is an automatic station designed to run without an operator. The turbine, built by the Cramp Company, is of 6,200 horsepower, and as far as we know this represents the largest unit which has so far been placed under automatic control.

Another plant of interest is the recent addition to the Spier Falls Station of the Adirondack Power & Light Corporation. This is an old station in which all of the original units were of the horizontal shaft type. The new addition represents a vertical shaft unit, marking the change in accepted practice which has taken place since the original installation. Figure 43 shows a cross section through the turbine. It will be noted that the runner is of the conventional type, the capacity being 9,000 horsepower, under a head of 80 feet. An interesting feature is the design of the volute casing, which is of circular transverse sections molded in the concrete. The drawing shows the method by which the casing reinforcement is tied in with the cast steel speed ring. Figure 44 shows a new construction of the spreading draft tube adopted in this plant proposed by H. B. Taylor, comprising separate cast stay vanes acting as columns at the discharge end of the spreading portion of the tube, these columns being imbedded in the concrete.

The rapid progress which has been made in applying the diagonal propeller type of unit to large installations is shown by the following installations undertaken during the year, including the Manitoba development mentioned above. These include the following:

- Manitoba Power Company, two units, 28,000 horsepower each, 56 feet head,
- Dryden Paper Company, one unit, 1,400 horsepower, 29 feet head,
- Spruce Falls Company, Ltd., one unit, 2,500 horsepower, 30 feet head,
- Anson Plant of the Great Northern Paper Company, at Madison, Maine, four units, 1,500 horsepower each, 20 feet head.

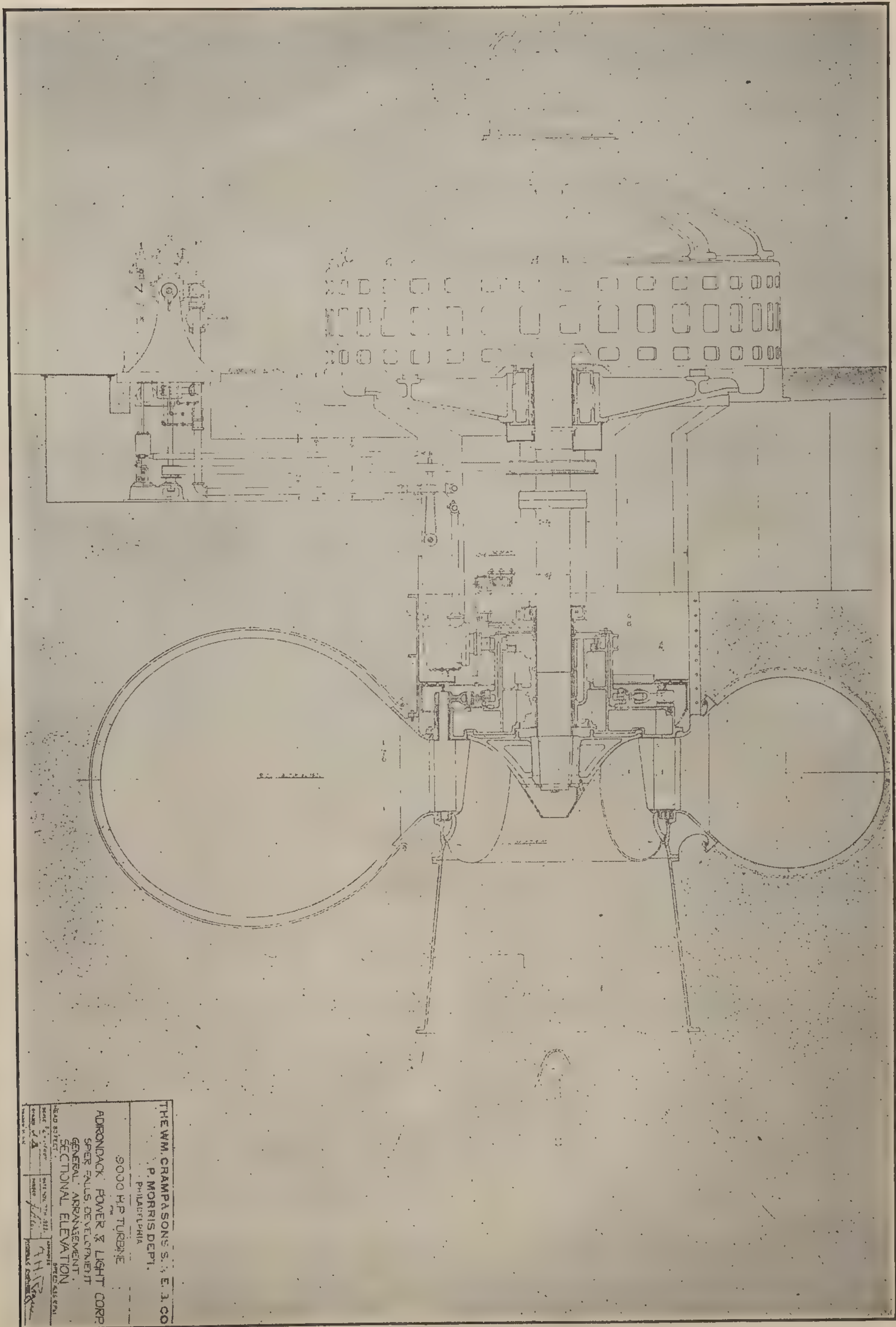


FIG. 43—SKETCH SHOWING GENERAL ARRANGEMENT OF 9000-HP. TURBINE FOR SPIER FALLS DEVELOPMENT, ADIRONDACK POWER & LIGHT CORPORATION.

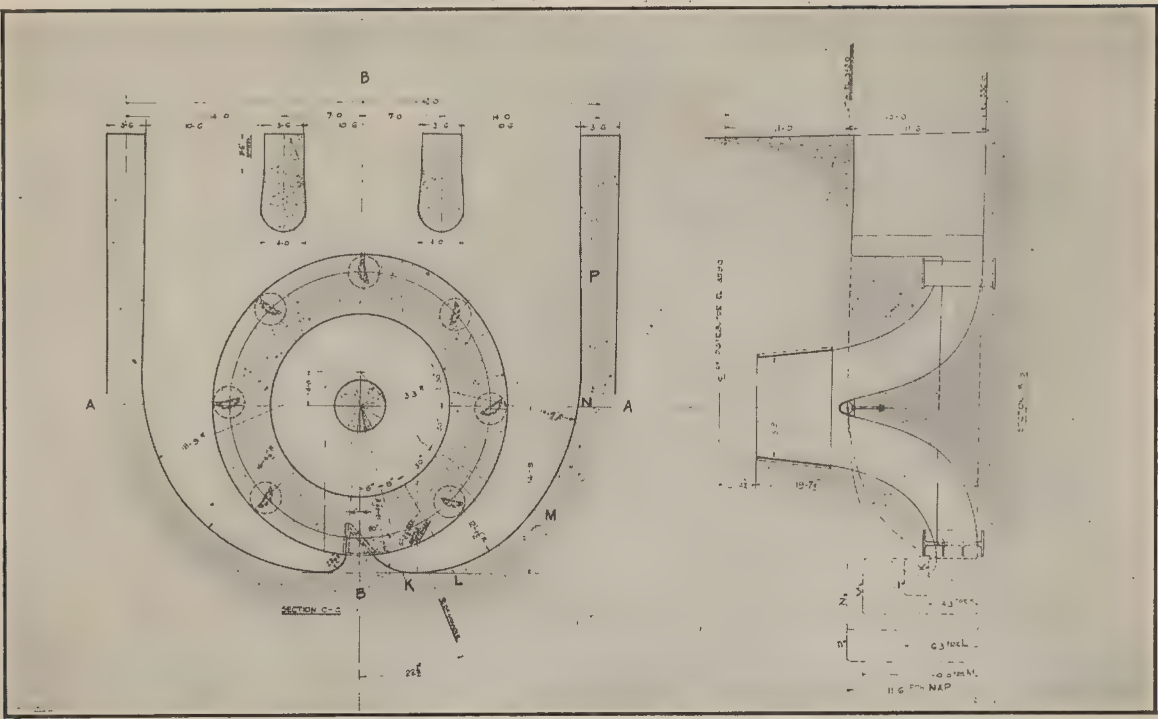


FIG. 44—SKETCH OF SPREADING DRAFT TUBE ADOPTED FOR SPIER FALLS DEVELOPMENT, ADIRONDACK POWER & LIGHT CORPORATION.

All of the above installations comprise I. P. Morris turbines, the first three representing contracts by the Dominion Engineering Works, Ltd., and the latter by the Cramp Company.

This year marked the placing of a contract with the Dominion Engineering Works, Ltd., for four additional I. P. Morris units for the Cedars plant near Montreal, which brings the total number of main units in this station up to eighteen.

There has been an opportunity in two installations to compare the performance of two otherwise similar units, one equipped with an elbow type draft tube and the other with a spreading draft tube. Such a comparison was made both in the Trenton Falls Station of the Utica Gas & Electric Company and in the Queenston Plant of the Hydro-Electric Power Commission of Ontario. In both installations the specific speed is low and therefore the velocity head to be regained by the draft tube is small at normal gate. As was to be expected therefore, the improvement in performance is more notable at part gate and overgate than at the point of highest efficiency. The results in the Trenton Falls Station have shown a definite improvement in maximum efficiency and a very material improvement in part-gate efficiency and a considerable increase in the full power output of the turbine. The final results are not yet available for the Queenston Station in regard to maximum efficiency and power, but a decided improvement in part-gate efficiency has been reported. Vacuum gauges connected to the draft tube show a substantial increase in vacuum with the spreading tube in both stations.

To show some of the types of construction used with the spreading draft tube, in addition to the sections of the Adirondack draft tube shown above, Fig. 45 is given illustrating the type of construction used in the I. P. Morris units furnished by the Dominion Engineering Works, Ltd., for the Maine-New Brunswick Power Company, the cone and spreading portion of the tube being of cast iron connected by stay bolts. Fig. 46 shows a plate steel construction used with the 1,400 horsepower turbine for the Dryden Paper Co., Ltd. The latter figure shows a simple form of turbine and setting for low head plants.

Fig. 47 shows a cross section through the 28,000 horsepower turbines for the Manitoba Power Company, mentioned above. The runners of these turbines are of the diagonal propeller type containing six blades and measuring nearly 16 feet in diameter over the blade tips. When the plant is finally completed, the head will be 56 feet. The turbines are designed for normal operation at this head at a speed of 138.5 r. p. m., corresponding to a specific speed of 153 in the ft.-lb. system, or 680 metric. The draft tubes are of the spreading type with the central core carried all the way up to the runner and made continuous in contour with the runner hub. This central core prevents the formation of a cavity in the upper portion of the draft tube water column under off-normal conditions of operation and improves the hydraulic conditions, and is also of mechanical advantage as it permits the turbine shaft to be lowered so that the lower end will rest upon the upper face of the cone, while erecting or dismantling the unit.

The structural arrangement of this installation is of interest. The weights of the upper portion of the unit are carried through separate stay vanes in the casing. Instead of being a part of the usual integral speed ring, these stay vanes are separately placed in the concrete and anchored by suitable reinforcement. Similar separate castings are used for the stay vanes at the lower end of the draft tube.

A valuable characteristic of the runner which has been found to be of great value in this installation is its ability to maintain a considerable power output at normal speed when the head is reduced to a low value, such as will exist in this plant for a considerable period before the entire development is completed. Owing to the high runaway speed of the propeller type turbine, it will operate and deliver power at heads so low that turbines of ordinary type would fail to operate at all, as such a condition would be beyond their runaway speed. As it will be about a year before the full head is available at the Manitoba plant, this characteristic of the diagonal propeller type turbines will secure to the Power Company

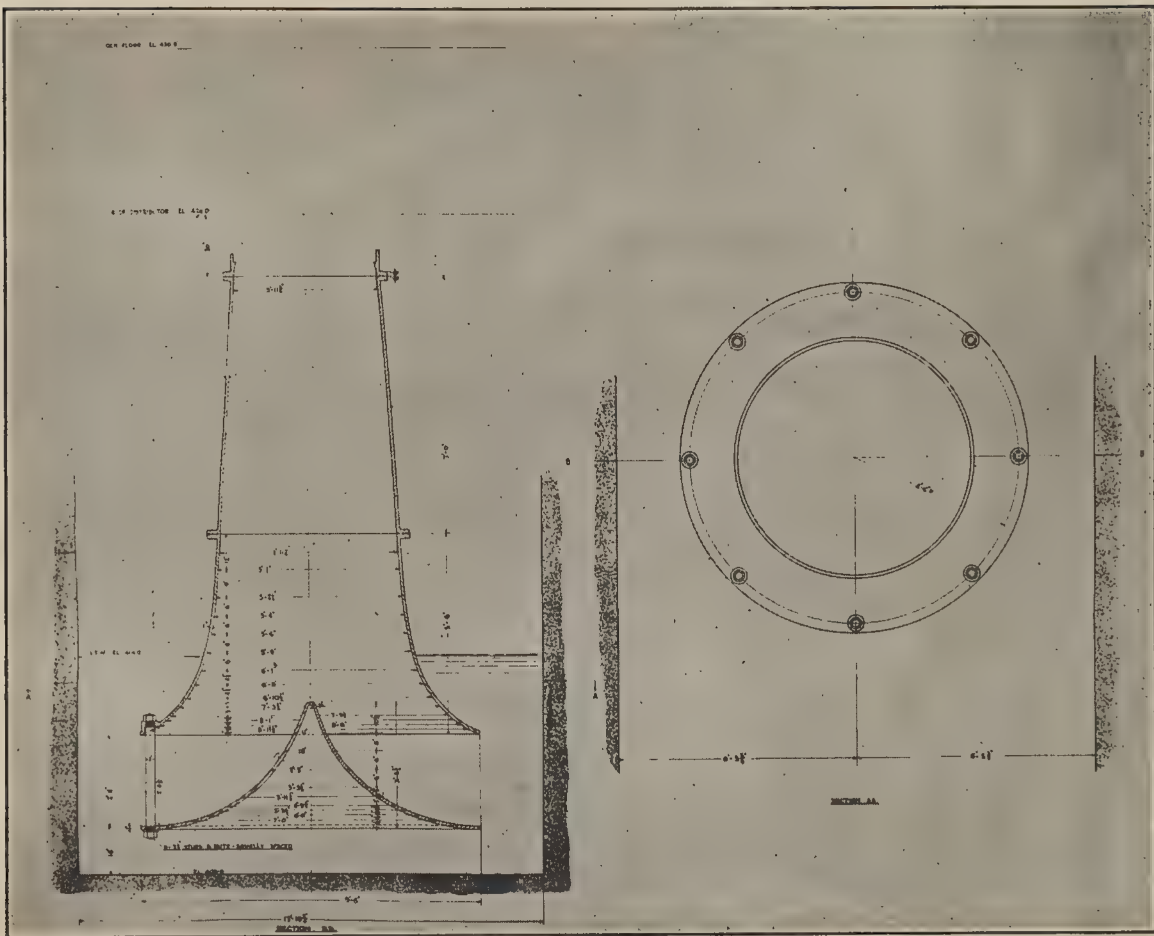


FIG. 45—SKETCH OF SETTING OF 2000-HP. TURBINE WITH CAST IRON SPREADING DRAFT TUBE FOR MAINE AND NEW BRUNSWICK ELECTRIC POWER COMPANY.

a large block of power which would have been unavailable with the conventional type. The first of these units is now in operation under a head of from 20 to 22 feet and is actually developing a specific speed of about 225 in the ft.-lb. system or 1,000 metric. The operation is reported to be extremely steady and without vibration and the turbine has proved to be easily controlled and capable of being started at a small gate opening. When first started under an effective head of between 17 and 20 feet, a gate opening of about 25 per cent was required to overcome the friction of the unlubricated surfaces of the Kingsbury bearing. After being once put into operation, it was found that the turbine could be restarted at from 3 per cent to 9 per cent gate opening. Tests made of this unit on water box resistance at a speed reduced to correspond with the normal speed under the designed head have indicated that the stepped-up results from the model tests will be fully attained. When the final head is reached in this plant, the average elevation of tailwater will be about 3 feet below the bottom of the runner. This low elevation of the turbine is an excellent safeguard against unstable hydraulic conditions in the draft tube.

Among the features of particular interest which will be incorporated in the two 70,000 horsepower turbines for the Niagara Falls Power Company may be mentioned the new method of sectionalizing the casing, introduced by H. B. Taylor, to permit its construction to be simplified and the separate sections to be more dependably bolted together. These casings will be of cast steel and there will be no separate speed ring or stay vane ring, the stay vanes being incorporated in the radial sections which are sub-divided so that the stay vanes form a part of the inner divisions, the outer portions of these radial sections being cast separately. The spreading type of draft tube will also be used with these units, and a number of recent improvements in details will be adopted including a new design of operating gear, an improvement due to F. H. Rogers, involving offset levers and compression links; adjustable lignum vitæ guide bearing for the main shaft; automatic system of governor control by which a change-over from governor to hand control is accomplished from a single central lever by means of fluid pressure; and a central lubricating system operated by compressed air.

The year has witnessed the adoption of valves of the Johnson type for turbine penstocks in a number of large installations. The Johnson valves for the 70,000 horsepower Niagara turbines will be the largest ever constructed.

The year has marked the resumption of construction work at the United States Government Plant at the Wilson Dam, or Muscle Shoals, and the completion of the turbines for this plant. These turbines, it will be remembered, will be of 30,000 horsepower capacity each and will operate under a head of 95 feet.

Experimental research has been continued in the field of hydraulic turbine development and among other achievements may be mentioned the progress made in improving the efficiencies of the high-speed propeller type turbine. The following efficiencies were given by Moody Diagonal Propeller type runners at the Holyoke flume:

88.7% at a specific speed of approximately 150 ft.-lbs. or 670 metric;

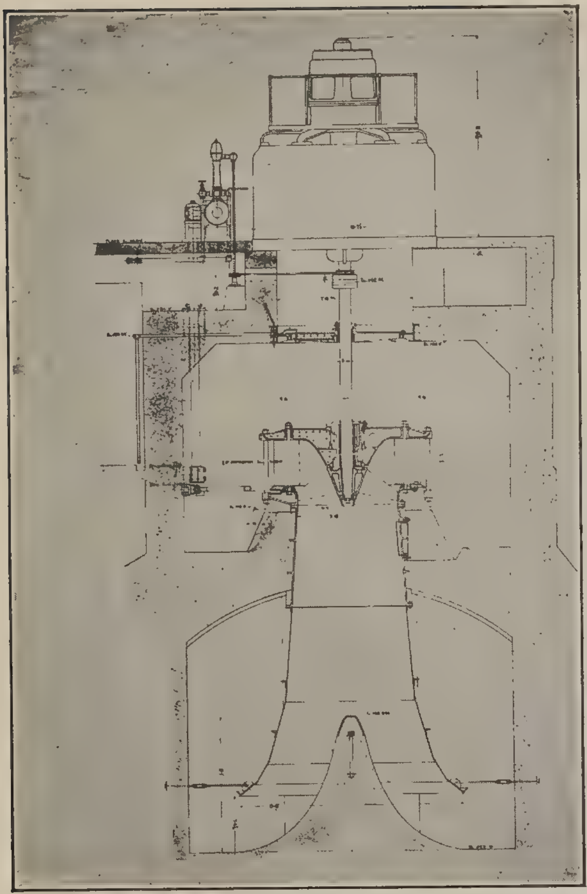


FIG. 46—SKETCH OF SETTING OF 1400-HP. TURBINE WITH PLATE STEEL SPREADING DRAFT TUBE FOR DRYDEN PAPER COMPANY, LTD.

90.8% at a specific speed of approximately 110 ft.-lbs. or 500 metric.

Considerable progress has also been made in the development of new types of impulse turbine which show the possibility of great increase in specific speed in the impulse field.

The general trend in unit sizes in the hydraulic turbine field has still continued toward the use of larger and larger units. The average capacity of units built by the Cramp Company during the year 1922 was 17,000 horsepower.

STATEMENT BY WELLMAN-SEAVER-MORGAN COMPANY

Our most recent turbine practice can be brought out by a brief description of the three 35,000 hp. single vertical reaction turbines which were awarded to us the first of March last year, by the Southern California Edison Company to go into their Big Creek No. 3 Development (See Fig. 48). This Plant is to operate under a head of 810 ft. at the start and this head will be drawn down to 740 ft. when all six units are installed and operated.

Provision is made in the design of the turbine and generators so that the frequency can easily be changed at any future date to 60 cycle when the units will run at 514 r. p. m.* The present speed for 50 cycles is 428 r. p. m.

The outstanding features of these turbines are the precautions taken to reduce leakage through gates and around runner, providing means for checking clearances around runner, the use of oil bearing of the babbitted type with two pumps, one belt driven and the reserve pump motor driven, efficient water and oil seal below bearings, splitting the cast steel spiral casing in two sections only, speed ring formed in casing, removable draft tube for dismantling runner and gates, easily replaceable breaking elements in the gate mechanism designed to let go before any other part reaches the elastic limit either upon closing or opening movement of gates, motor driven load limiting device on the Woodward governors, high pressure lubricating system, and combined control column near governor. The runners are made of special manganese bronze having high tensile strength, exceptionally hard surface and high resistance against corrosion. The draft tubes have welded steel plate liners from the removable cast iron section down to the bottom of the bend and the liners are stiffened with Z-bars which also anchor them into the concrete. The lower part of tubes are formed in concrete according to a very efficient design developed during the past year, the forms for which are very easily constructed, mostly made up of straight sections and requiring no fillets in the corners.

These units are the most powerful ever built for operation under such a high head and mark another distinct step forward.

The subject of draft tubes is of great interest to the hydraulic engineering profession and plays a very important part on every new development. The work which the Alabama Power Company completed last year at Worcester Polytechnic Institute in testing and research work on the different types of tubes will be of great value, since all factors have been reduced to a common denominator and a true comparison obtained. We presume some sort of resume covering this work will be published. Fig. 48 shows our new type of tube as designed and constructed for the Big Creek No. 3 Plant of the Southern California Edison Co., mentioned above, and Fig. 49 shows the steel plate draft tube as installed at the Union Gas Plant of the Central Maine Power Company. This last tube can be used to advantage where it is not possible to work in a concrete form of draft tube, also in remodeling old plants it permits of a decided reduction in velocity in the vertical leg of the draft tube for a given draft head without unduly increasing the spacing between units and the depth of excavation.

We have been testing draft tubes continuously since May 1, 1922, and have brought out many novel designs. We were also called upon to make a couple of tests of the conduit spillway or head increasers which are to be installed on two of our turbines at the Alcona Development of the Consumers Power Company. All of this information will be published as soon as we can get it into proper shape.

Instead of driving the flyballs of the governor by belt or by gearing, the Woodward Governor Company has developed a motor drive for this purpose which has proved successful on a couple of developments, the largest of which has just been installed on two 4000 h.p. turbines of our make under 52 ft. head for the Lake Superior District Power Company near Tony, Wisconsin. The synchronous motor driving the flyballs is tied in to the armature of the particular generator which is to be governed.

We have installed several of our new needle type valves the past year, which upon testing under the most severe permissible conditions were decidedly successful. In several cases the valve was used to shut off full discharge of turbine with gates wide open. The closures were made under these conditions and the plungers were under perfect control at all points of the stroke. At the end of the stroke, the plungers would come up to their seat positively and without any semblance of slamming or chattering. In one case, the velocity through the valve was 33 ft. per second. With turbine casing empty and turbine gates open, the valves have also been used to start up the machine without any by-pass whatsoever and they could be opened positively to any desired position. The action of the plungers in all installations was dead beat and no creeping or overtravel could be measured. With the control valve eliminated by closing cocks in the connections to main valves, the plunger would still remain inert in any desired position, thus demonstrating the efficiency of the balancing device.

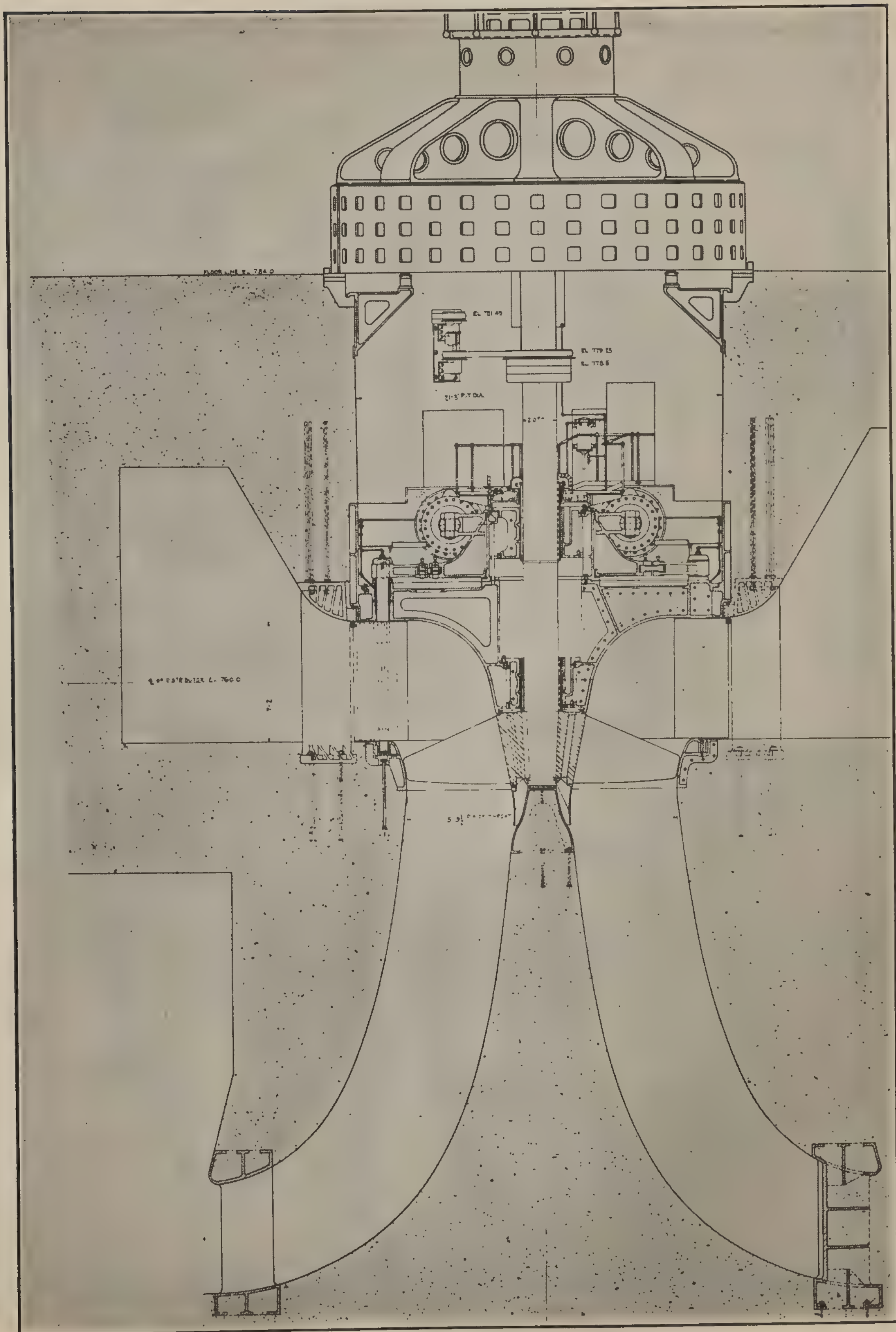


FIG. 47—SECTIONAL ELEVATION OF 28,000-HP. TURBINES FOR MANITOBA POWER COMPANY, LTD.

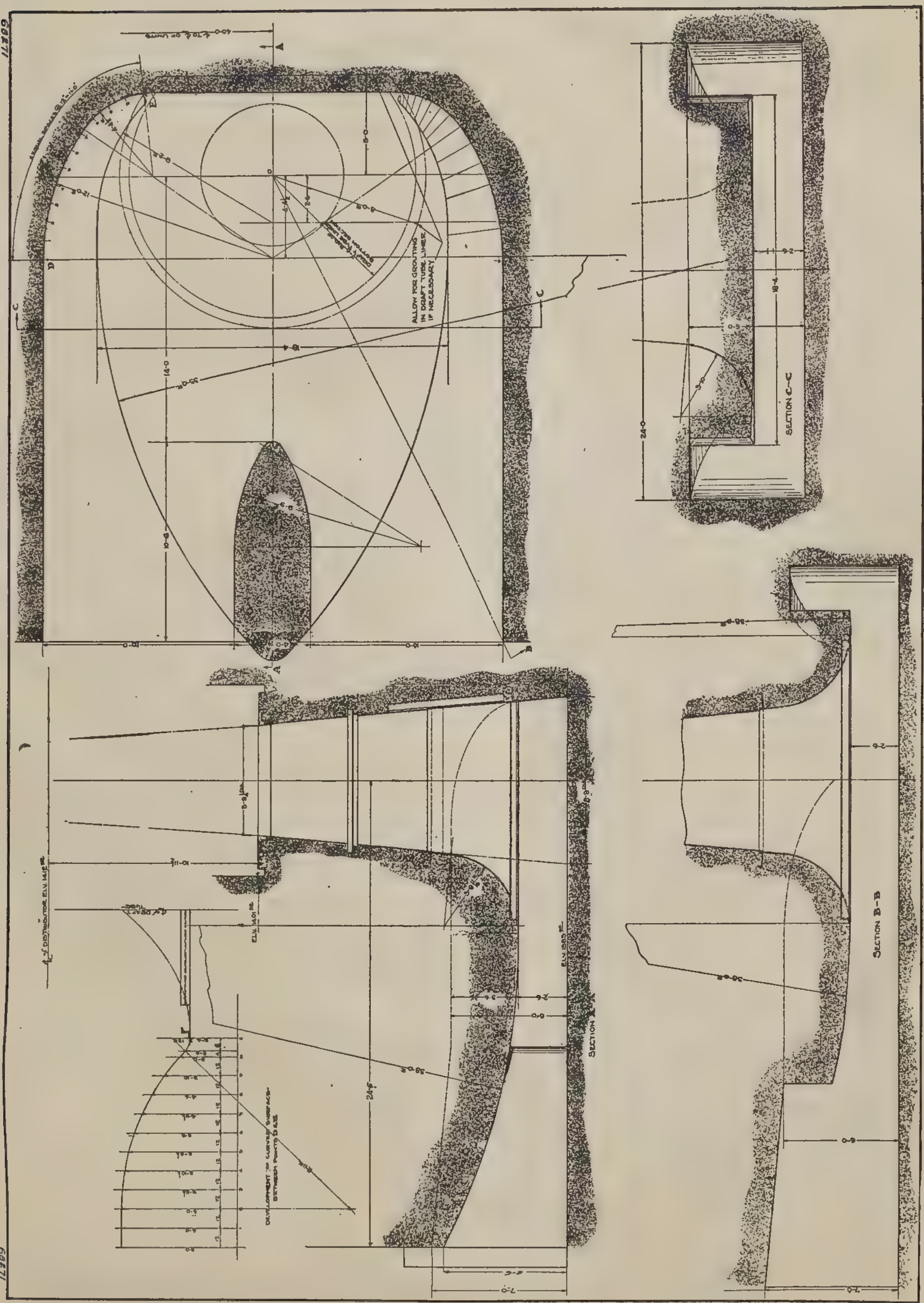


FIG. 48—SKETCH OF DRAFT TUBE FOR BIG CREEK NO. 3 DEVELOPMENT OF SOUTHERN CALIFORNIA EDISON COMPANY.

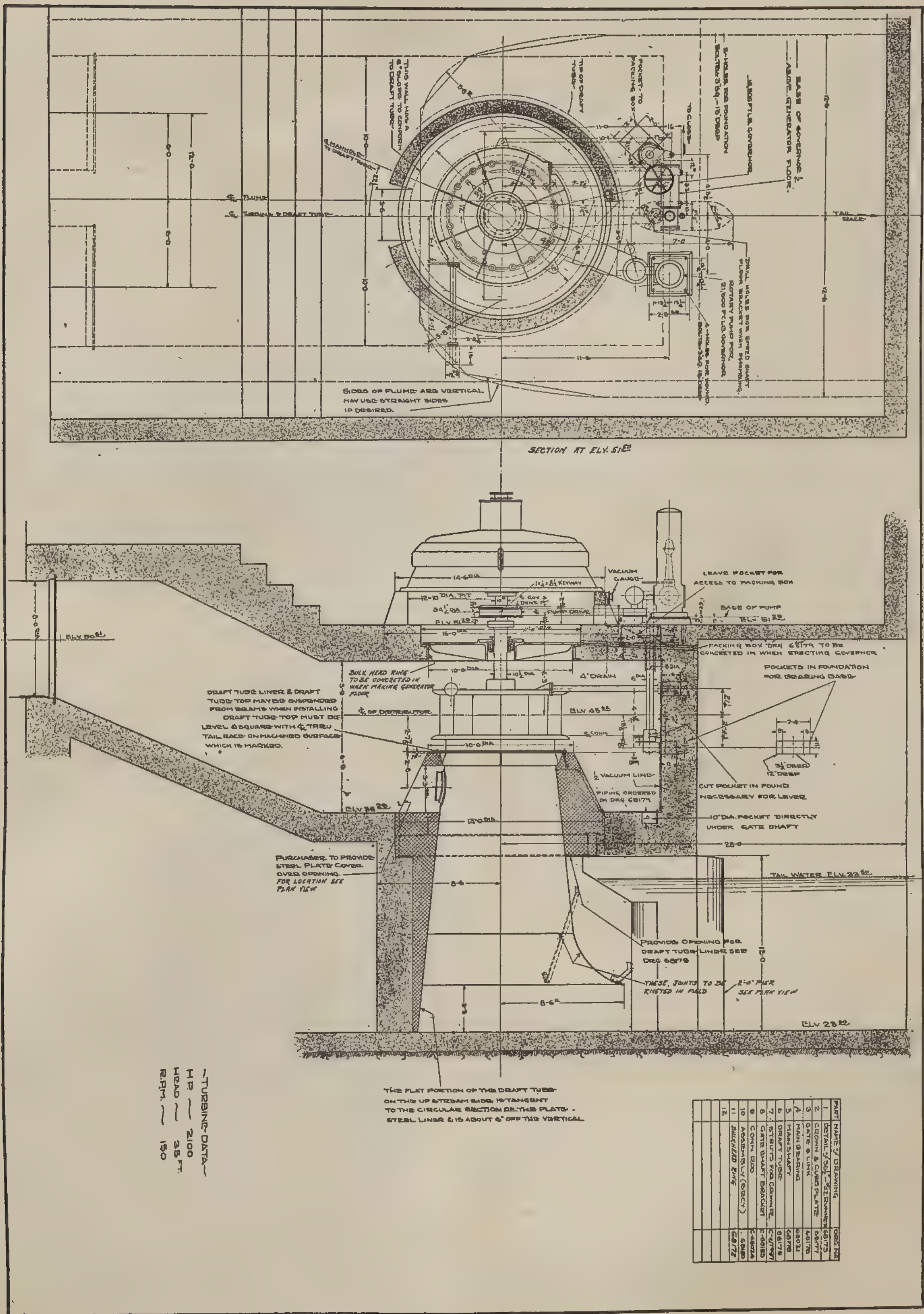


FIG. 49—SKETCH OF REMODELED SETTING OF TURBINE WITH STEEL PLATE DRAFT TUBE FOR CENTRAL MAINE POWER COMPANY.

Some of the exceptional installations are the three 54-in. cast steel valves for the Southern California Edison Company under 810 ft. head, two 42-in. valves under 250 ft. head for the San Fernando Plant, Los Angeles, two 30-in. valves under 2200 ft. head for Western States Gas & Electric Company, and two 56-in. valves for Northern New York Utilities under 250 ft. and one of the same size valves for the New England Power Company's Searsburg Plant under 240 ft. head. The above valves are all hydraulically operated. Two 48-in. mechanically operated valves were also installed at the Shoshone Project of the U. S. Reclamation Service.

We not only have a flood of inquiries for new development work, but notice a surprising tendency to remodel old existing plants to increase their efficiency and to reduce the cost of maintenance and repairs.

STATEMENT BY ALLIS-CHALMERS MANUFACTURING COMPANY

During the year 1922 we had under way one of the largest building and construction programs which we have ever experienced in hydraulic turbine work. At one time we had under contract and in the process of erection over three-fourths of a million horsepower in hydraulic turbines. Our new business for the year 1922 totaled 431,110 hp. It would be difficult to state just how much capacity was installed during the year 1922 because some plants were started that were largely erected during the preceding year and also because plants now in the process of installation will not be formally started until next year. Roughly, we might state that we have water wheels of an aggregate capacity of 400,000 hp. in the process of installation. Approximately the same amount might be stated to have been installed during the year 1922.

Power House Arrangement

The general arrangement of a power house unit which seems to find greatest favor for medium heads consists of a single vertical plate steel spiral cased turbine with pressure regulator, hydraucone and integral type governor. The most noteworthy higher head installation of this type of arrangement is the Davis Bridge Plant of the Power Construction Company, for whom two 20,000 hp. units are being constructed for a head of 345 ft. It is significant to note that to date the Allis-Chalmers Manufacturing Company has installed over sixty major installations of units involving the plate steel spiral casing construction, these sixty installations totaling nearly 1,100,000 hp. This type of unit is particularly attractive for export work on account of the ease with which the casing may be handled. We have built several 18,000 hp. turbines of this type for Japan.

Record Capacity of Units

The most noteworthy new contract entered into was that covering the 70,000 hp., 65,000 kv-a. direct connected unit for the Niagara Falls Power Company. This unit is of the plate steel spiral cased vertical shaft type and is designed for a speed of 107 r.p.m. under 123 ft. head. This casing inlet will be 15 ft. in diameter. The largest units started in the states were those in the Mt. Shasta Pit River No. 1 plant of the Pacific Gas & Electric Company (California). These units were designed for 421 ft. head and they have carried in excess of 45,000 hp. each on commercial load. They are of the cast steel spiral cased type.

Butterfly Valves

With the Niagara Falls unit we have received an order for the three largest valves ever constructed. These are of the butterfly type 23½ ft. in diameter. They are designed for 60 lb. pressure. Each valve weighs nearly one-half million pounds. Its disc is designed to carry a total load of 3,750,000 lb.

Recent tests by owners and users of both types have confirmed our belief that this type of valve with its straight streamline flow has smaller loss of head than possible with the various types of needle valve.

High Speed Runners (Suction Type)

During the past year the majority of our low head installations have been built using the high-speed suction type of runner usually built with four blades. There has been little or no change in general arrangement of this type of unit from that used during the preceding five years. This type effects such a perfect solution of low head developments that several other builders are evidencing a desire to break into this field. To the best of our knowledge, no plants other than those we have constructed have been placed in operation for their rated conditions of head and capacity.

The general conditions leading up to the invention and development of the high-speed type of runner have been described fully in the transactions of the American Society of Mechanical Engineers for December, 1919. Since the initial announcement in 1919 there has been put into operation hydro-electric units for heads as high as 55 ft., for discharge capacities comparing favorably with those of the largest runners in the world, and in sufficient number to indicate that for low heads the mixed flow type (Francis) is rapidly being superseded.

The fundamental reason why the mixed flow runner does not operate advantageously under low head conditions is due to the fact that considerable of the flow through it is in a radial inward direction. With the relatively higher speed necessary to maintain synchronism under reduced head, the relatively higher centrifugal forces in the water passages through the runner oppose the flow through it and consequently reduce its discharge and horsepower. The high-speed design of turbine approximating straight radial blades practically eliminates this inward flow with its large centrifugal force, with the result that at reduced head occurring in flood conditions its discharge and horsepower per foot of head are very largely increased over those at the normal head.

Some novel applications of the high-speed runner principle have been made in connection with direct connected units such as water-wheel-driven pumps for irrigation purposes, the high speed permitting of the elimination of gears.

The most noteworthy accomplishment in the high-speed runner field was the successful starting up of the Green Island plant of Henry Ford & Son on the Hudson River near Troy, N. Y. (See Fig. 50 showing runner and Fig. 51 showing interior of generator room.) This plant is being described fully in a current issue of *Power*. The units have been given capacity and regulation tests at all gate openings and for heads varying from 9 ft. to 16 ft., necessitating specific speeds sometimes considerably in excess of 200. These tests showed the units to be in every way acceptable.

Considerable additional test data and power plant data have been obtained, bearing on the limit of heads to which the suction type of runner is adapted. These data lead us simply to repeat and emphasize our statement of last year:

> "Based on all data available, it is the present opinion of the Allis-Chalmers Company that specific speeds in excess of 150 English System should not be utilized for heads greater than 30 ft."

A more definite explanation of this limit is intimated in the name given to this runner type. It is called the suction type because power is actually developed by reason of a suction or under pressure action rather than by a pressure action. Oddly this means that most of the power is actually produced on the back side of the blades. This feature is strictly analogous to the performance of an airplane wing in which the major portion of the lifting force is developed by under pressure on the upper surface of the wing, whereas only a minor portion is developed by a pressure from below. This suction principle involves two radical departures in turbine design. In the first place it is the back side of the runner blade that is designed to give the desired power and efficiency performance, the front side being a secondary consideration. The second feature involves a departure from some accepted rules of turbine computations, one of which is that the power of a

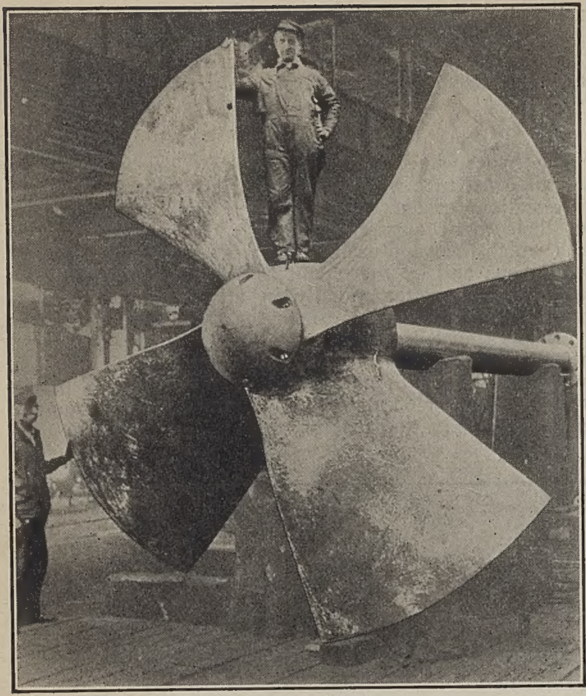


FIG. 50—156-IN. DIAMETER 2200-HP. FOUR-BLADE CAST STEEL NAGLER RUNNER, CAST IN FOUR SECTIONS, WEIGHT 17,000 LB., HEAD 13 FT., SPEED 80 R.P.M.—FOR HENRY FORD & SON, INC., GREEN ISLAND PLANT.

runner varies as the three halves power of the head. This law does not strictly apply because there is a definite limit to the atmospheric pressure available for producing the suction effect, so vital to the horsepower of these runners of high specific speed. This feature has been very precisely pointed out by W. M. White, who calls attention to

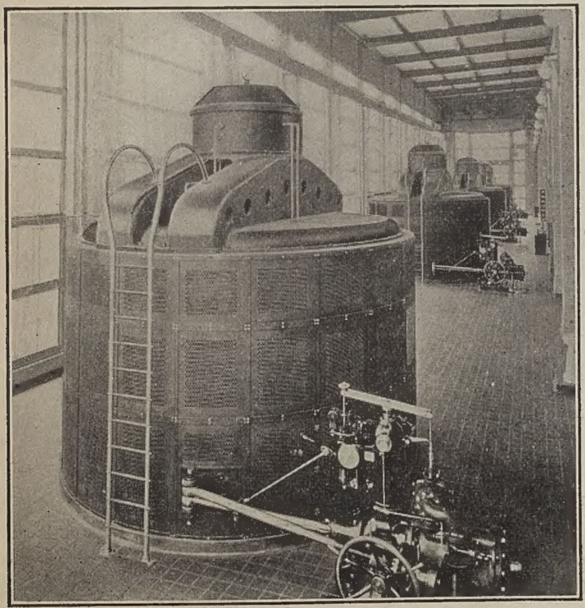


FIG. 51—FOUR 800-K.V.A., 4600-VOLT A.C., 1000-KW., 250-VOLT D.C., 80 R.P.M. DOUBLE GENERATORS DRIVEN BY 2200-HP. 13-FT. HEAD TURBINES—HENRY FORD & SON, INC., GREEN ISLAND PLANT.

the fact that it is not sufficient merely to keep the sum of the static draft head and the velocity head at the discharge of the runner a reasonable amount below 34 ft., but that a third factor must be introduced, this third factor being a large percentage of the total head. Practically speaking, this means that high specific speeds, for example in excess of 150, are not readily obtainable with heads in excess of 30 or 34 ft., particularly if the runner is placed at or above tail level.

Numerous units are at present under construction varying in capacities up to several thousand horsepower each for heads from 7 ft. to 32 ft. and for runner diameters up to 10 ft.

Additional increases in efficiency have been effected since last year. The original runner which showed 87 per cent at Holyoke in 1916 has been so improved that 10-in. models of the original runner and the newest design show an increase of 5 per cent when tested under 12-ft. head. Holyoke check on this increase is being obtained.

Impulse Wheels

The past year has been characterized by a greater volume of large impulse wheel business than this company has ever before faced. Five units totaling over 100,000 hp. capacity were on our erecting floor at one time. No new world's records for capacity have been established since the installation of the 30,000-hp. impulse wheel units in the Caribou plant of the Great Western Power Company. Impulse units of 70,000 hp. capacity each are in the preliminary stages of design.

A new line of impulse wheel development may be expected in the near future. We have been experimenting on this for several years and the work is sufficiently advanced to permit of its announcement to the public and to permit of our undertaking the building of units for moderate heads and capacity embodying the new principle. Complete description of this line of work was contained in a paper presented by Forrest Nagler before the Milwaukee section of the American Society of Mechanical Engineers. Significant paragraphs from this paper are as follows:

> The type of impulse wheel now universally adopted is known as the Pelton Wheel, the Tangential Wheel, the Pressure-less Wheel or, more generally, the Impulse Wheel. This type, a distinctively American product, perfected by the men of the California mining industry, has superseded all other types for high head work.
>
> It is characterized by:
>
> First—A free jet operating under the full spouting velocity due to the operating head.
>
> Second—Substantially tangential application of the jet to the wheel.
>
> Third—A bucket velocity practically 50 per cent of the jet velocity.
>
> Fourth—Splitter type buckets concave on the working surfaces.
>
> The design developed by the author conforms only to the first item, a requirement inherent in any impulse wheel. Departure from item two is radical, as, in the new high-speed types, the jet must always make a large angle with the tangent, hence the name X Flow. The result of this X Flow is to permit of relative wheel velocities comparable with those obtainable with reaction wheels. Wheels have been built and tested having bucket velocities twice and four times those of the present types. It is shown that with the new type this speed may be increased indefinitely without violating the principles of hydraulics, although mechanical and frictional limitations may introduce restrictions.
>
> The main conclusion is this: Speed in impulse wheel work is a function of the angle the jet makes with a tangent.
>
> The main purpose of this paper is to forcibly draw attention to the possibility of departing from the orthodox tangential flow and 50 per cent coefficient, departing even from the specter of practice, precedent,

usage and text books, without violating perfectly sound hydraulics.

It is believed that the new type will not only serve to fill a gap that cannot be covered by desirable arrangements of either reaction or impulse wheels, but will ultimately take over a material portion of the field now developed, more or less disadvantageously, with the accepted types of impulse or reaction wheels.

Governors

The general tendency on governors is in the direction of simplification, and they are being more and more merged into the structure of the turbine proper. The Allis-Chalmers Manufacturing Company has standardized for about 20 years on the rotary type of oil pump in spite of almost unanimous opposition from both turbine and governor builders. The last "conscientious objector" has apparently passed, as all builders have recently come to adopt this type. Of the hundreds of installations made by this company not more than three, and those were of the earlier designs, have given trouble sufficient to necessitate repair or replacement. To date 53 major installations have been made using the distinctive feature of direct connected flyballs. These installations total 890,000 hp.

Draft Tubes

The hydraucone regainer represents the most efficient type of draft tube with which we are familiar. Numerous modifications of curved tube are making their appearance, but their merit apparently is measured by the closeness with which they approach the narrow throat height of the hydraucone and by the extent to which they provide get-away for the water radially in all directions from the centerline. The number of these tubes that are making their appearance is the finest tribute that we can cite to the correctness of the hydraucone principle. To date hydraucones have been installed by this company in connection with over 60 major capacity units, totaling approximately 865,000 hp. in capacity.

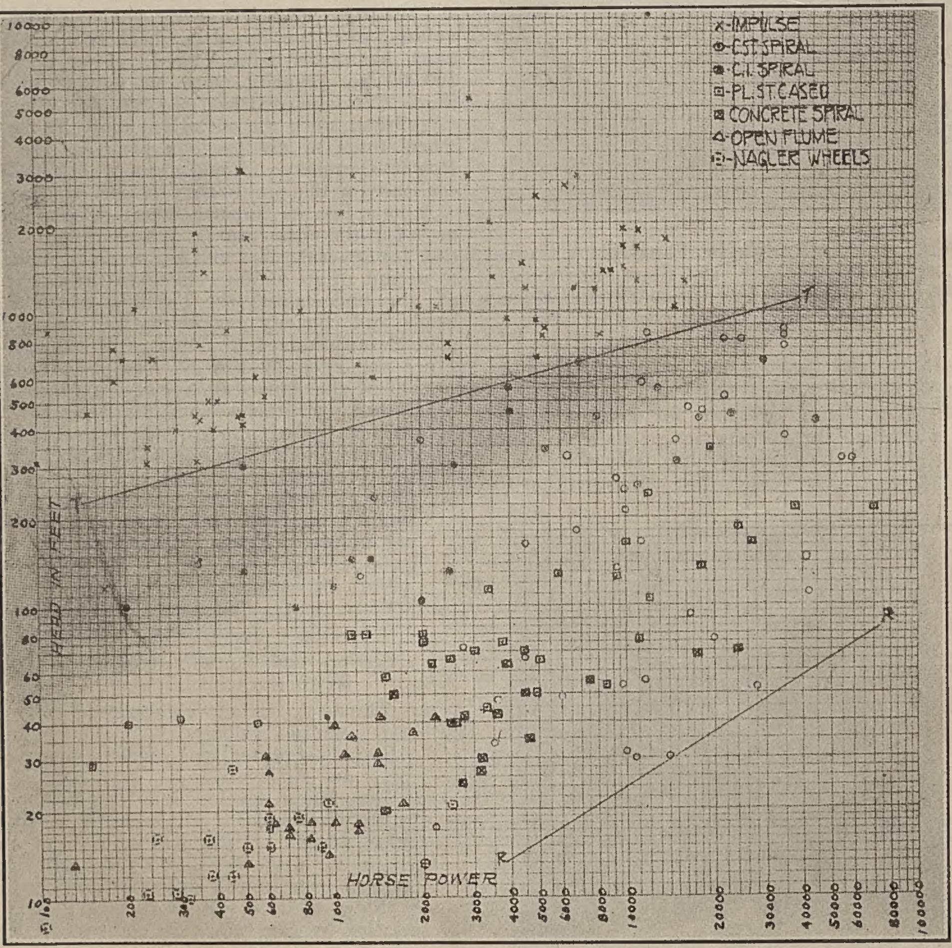


FIG. 52—A PLOTTING OF THE ENTIRE FIELD OF HYDRAULIC TURBINES, ILLUSTRATING PRACTICALLY ALL OF THE NOTEWORTHY PLANTS EVER BUILT. THE OUTLYING PLOTTER POINTS REPRESENT WORLD RECORDS EITHER IN RESPECT TO SIZE, EXTREMES OF HEAD, EITHER HIGH OR LOW, CAPACITIES, ETC. THE POINTS ABOVE THE LINE TT REPRESENT IMPULSE UNITS, PRACTICALLY ALL OF THEM BEING OF THE TANGENTIAL TYPE. THE LINE RR REPRESENTS APPROXIMATELY THE LIMIT OF REACTION UNITS EITHER IN SIZE OR CAPACITY FOR MODERATE AND LOW HEADS, THIS LIMIT BEING FIXED BY COST, SHIPPING DIMENSIONS OR GENERATOR SPEED.

A SHADED PORTION LARGELY BELOW BUT ADJACENT TO THE LINE TT REPRESENTS A FIELD LYING BETWEEN THE TANGENTIAL IMPULSE AND THE HIGH HEAD REACTION FIELD, WHICH IS AT PRESENT DEVELOPED DISADVANTAGEOUSLY WITH THESE TWO TYPES. IT REPRESENTS THE FIELD TO WHICH THE PROPOSED TYPE OF X FLOW WHEEL IS PARTICULARLY ADAPTED. IMPULSE WHEEL HORSEPOWERS ARE PLOTTED FOR ONE JET.

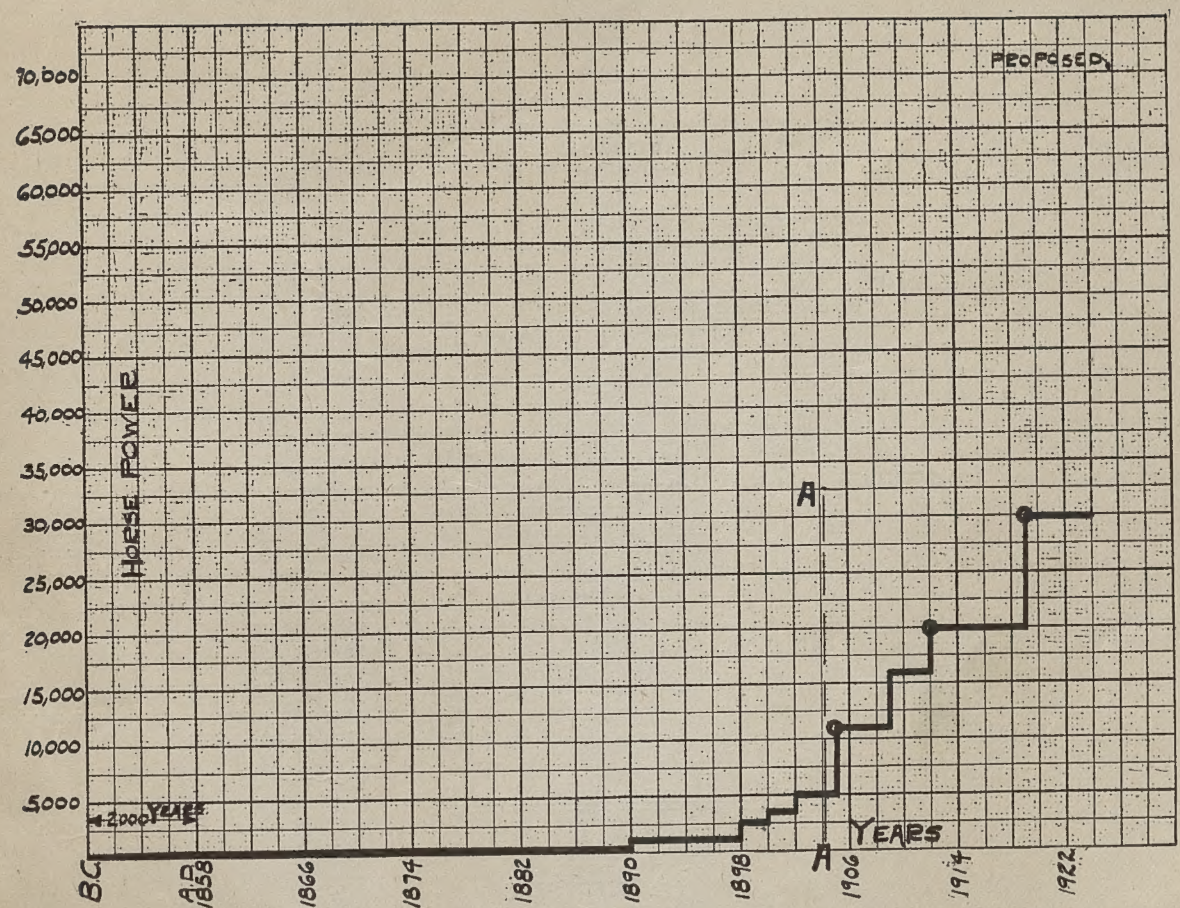


FIG. 53—CURVE SHOWING THE REMARKABLE RECENT DEVELOPMENT IN CAPACITY OF IMPULSE UNITS. THIS MIGHT BE DETERMINED A SCIENTIFIC PROGRESS CURVE OF MANKIND. IT ILLUSTRATES FORCIBLY THE EXTREMELY SLOW PROGRESS MADE FOR OVER 2000 YEARS IN CONTRAST TO THE RAPID GROWTH AND DEVELOPMENT WITHIN THE LAST 30 YEARS. POINTS MARKED WITH A CIRCLE REPRESENT PIONEER OR RECORD CAPACITY MACHINES BUILT BY ALLIS-CHALMERS MANUFACTURING COMPANY

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